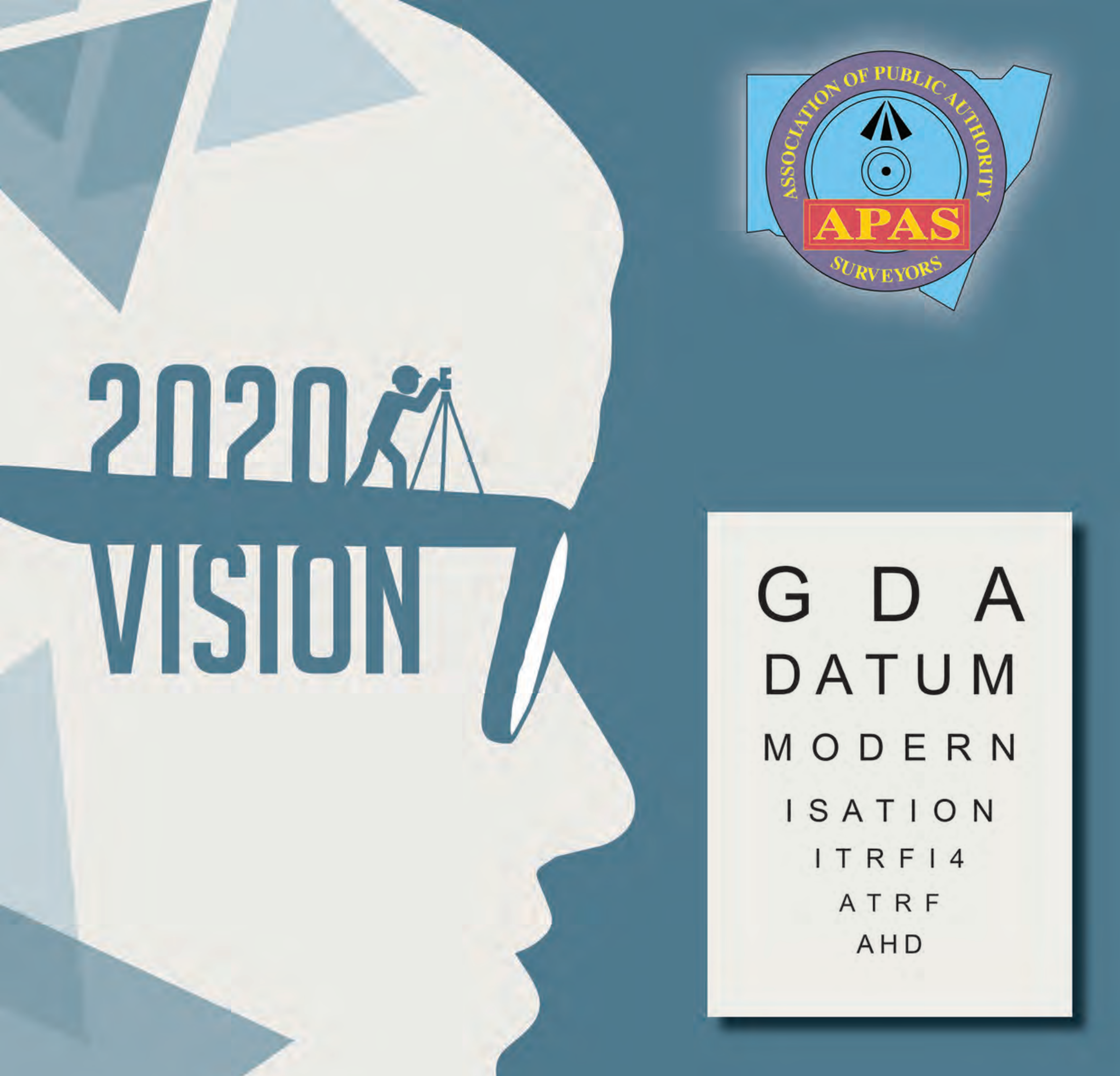

Proceedings of the APAS2017 CONFERENCE

Ramada Resort, Shoal Bay, **20-22 March 2017**

Edited by Dr Volker Janssen

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Editorial

These proceedings contain the papers presented at the Association of Public Authority Surveyors Conference (APAS2017), held in Shoal Bay, NSW, Australia, on 20-22 March 2017. Papers were not peer-reviewed but have been subject to changes made by the Editor. The Editor would like to thank all authors for their contributions covering a wide range of topics relevant to the surveying and spatial information community, thus ensuring an exciting and informative conference.

Authors are welcome to make their paper, as it appears in these conference proceedings, available online on their personal and/or their institution's website, provided it is clearly stated that the paper was originally published in these proceedings. Papers should be referenced according to the following template:

Janssen V. (2017) GDA2020, AUSGeoid2020 and ATRF: An introduction, *Proceedings of Association of Public Authority Surveyors Conference (APAS2017)*, Shoal Bay, Australia, 20-22 March, 3-20.

Spatial Services, a unit of the NSW Department of Finance, Services and Innovation (DFSI) is gratefully acknowledged for providing the front cover artwork and producing these proceedings. The APAS committee is thanked for their hard work in organising this conference.

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GDA2020, AUSGeoid2020 and ATRF: An Introduction

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ABSTRACT

The Geocentric Datum of Australia 2020 (GDA2020) is a new, much improved Australian national datum that will replace the current GDA94 by 1 January 2020. This new datum is to be used in conjunction with the new AUSGeoid2020 in order to connect to the Australian Height Datum (AHD). By 2020, GDA2020 will be complemented (and eventually replaced) by the time-dependent Australian Terrestrial Reference Frame (ATRF). This paper provides some background on coordinate systems and datums before explaining these terms in general. It outlines how important this change is for users intending to benefit from the improved geodetic infrastructure. The efforts undertaken by DFSI Spatial Services to support this datum modernisation in New South Wales are also summarised.

KEYWORDS: *Datum modernisation, geodetic infrastructure, GDA2020, AUSGeoid2020, ITRF, ATRF.*

1 INTRODUCTION

The Geocentric Datum of Australia 1994 (GDA94) has been our national datum since its adoption in 2000, providing fundamental positioning infrastructure for Australia (ICSM, 2014). Vertical coordinates are referred to the Australian Height Datum (AHD) (Roelse et al., 1975). Significant improvements in positioning technology in the recent past now enable centimetre-level positioning capability via Global Navigation Satellite System (GNSS) techniques such as Network Real Time Kinematic (NRTK) and Precise Point Positioning (PPP) (e.g. Janssen and Haasdyk, 2011; Rizos et al., 2012), while decimetre-level accuracy or better will soon be available to the mass-market. These developments have revealed that GDA94 is not capable of providing the required quality of datum into the future. Consequently, Federal and State Governments have worked towards modernising Australia's datum for some time (ICSM, 2016a).

Datum modernisation is required in order to accommodate the increasing accuracy and improved spatial and temporal resolution available from modern positioning technologies to an ever-broadening user base concerned with surveying, mapping, navigation, engineering, machine guidance and precision agriculture, to name but a few. The goal of datum modernisation is to supply all users with the most complete yet most straightforward datum products that can define a locally consistent set of coordinates, such that their positioning device agrees with the physical world and associated spatial data to an acceptable level of accuracy (Haasdyk et al., 2014b).

In this context, it is important to remember that geodetic control underpins *all* spatial data, including water, boundaries, addresses, utilities, transport, elevation and imagery, but also

that most revenue and GNSS data consumption will come in the near future from applications such as location-based services and transport (GSA, 2015, 2016).

The Geocentric Datum of Australia 2020 (GDA2020) is a new, much improved Australian national datum that will replace the current GDA94 by 1 January 2020. While the GDA2020 technical manual has not been released publicly at the time of writing, this is expected to happen soon. Figure 1 illustrates the difference between GDA94 and GDA2020, both in regards to the coordinate shift and the respective uncertainty. The effect of tectonic motion since 1994, resulting in a shift of approximately 1.8 m towards the north-east, and the improvement in coordinate quality are clearly visible.

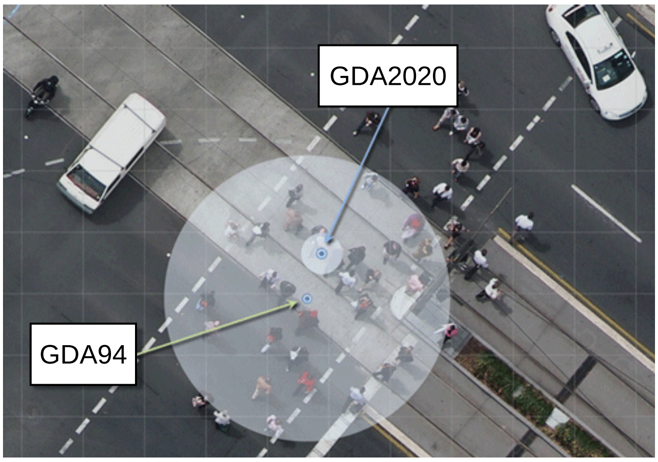


Figure 1: Artistic illustration of GDA94 and GDA2020 coordinates and their uncertainties (adapted from Jaksa, 2015).

GDA2020 is expected to be released soon and is to be used in conjunction with the new AUSGeoid2020 (released at the same time) in order to connect to AHD. By 2020, GDA2020 will be complemented (and eventually replaced) by the time-dependent Australian Terrestrial Reference Frame (ATRF).

This paper provides some background on coordinate systems and datums before explaining the terms GDA2020, AUSGeoid2020 and ATRF in general and outlining how important this change is for users intending to benefit from the improved geodetic infrastructure. The efforts undertaken at DFSI Spatial Services to support this datum modernisation in NSW are also summarised.

2 COORDINATE SYSTEMS AND DATUMS

A coordinate system (or coordinate reference system) is a methodology to define the specific location of a feature in space. As illustrated in Figure 2, positions on the ellipsoid are usually expressed in Cartesian coordinates (X, Y, Z) or curvilinear geographic coordinates (ϕ, λ, h), i.e. latitude, longitude and ellipsoidal height. In a geocentric, rectangular Cartesian coordinate system, the origin is the earth's centre of mass, and the Z-axis coincides with the position of the earth's rotation axis at a certain instant in time (epoch). The X-axis passes through the intersection of the Greenwich meridian and the equator, and the Y-axis completes a right-handed coordinate system by passing through the intersection of the 90°E meridian and the equator. In regards to curvilinear geographic coordinates, latitude is defined as the angle in the meridian plane between the equatorial plane and the ellipsoid normal through a point P.

Longitude is measured in the equatorial plane as the angle between the Greenwich meridian (*X*-axis) and the meridian through P, while the ellipsoidal height is measured from the ellipsoid surface along the ellipsoid normal. It is important to note that a single ground point can have different curvilinear coordinates depending on which ellipsoid the coordinate system refers to.

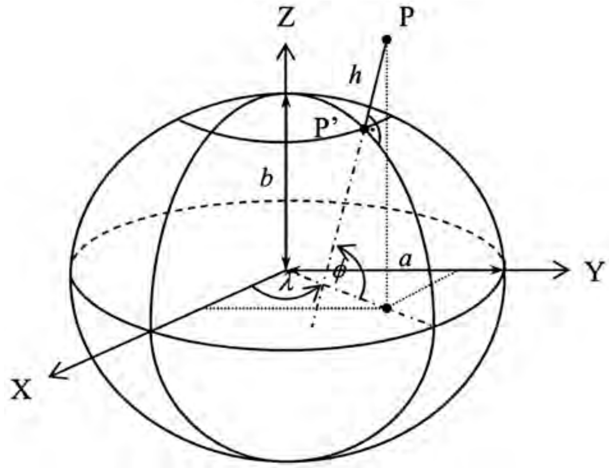


Figure 2: Ellipsoidal coordinate systems (Janssen, 2009b).

Since coordinate systems are idealised abstractions, they can only be accessed through their physical materialisation (or realisation) called reference frames or datums. The datum effectively defines the origin and orientation of the coordinate system at a certain epoch, generally by adopting a set of station coordinates. Over time, different techniques with varying levels of sophistication have been applied to define the shape of the earth's surface, resulting in the adoption of many different datums.

In practice, it is often required to express positions on a flat surface in the form of grid coordinates, i.e. in a 2-dimensional Cartesian coordinate system such as Easting and Northing. This is achieved by map projections according to a recognised set of mathematical rules, resulting in an ordered system of meridians (lines of constant longitude) and parallels (lines of constant latitude). The most common projection used in Australia is the Universal Transverse Mercator (UTM) projection, which utilises a zone width of 6° (Figure 3).

The UTM projection ensures that the scale is very close to unity across the entire zone by defining a central scale factor of 0.9996 for the central meridian (CM), resulting in a scale of 1.0010 at the zone boundary located 3° away from the CM. In order to ensure positive coordinate values across the entire zone, the UTM system applies false coordinates to the origin by adding 500,000 m to the true Easting and, in the southern hemisphere, 10,000,000 m to the true Northing. The conversion between curvilinear and grid coordinates was traditionally performed using Redfearn's (1948) formulae. However, now it is preferred to use the much more accurate Karney-Krueger equations (e.g. Deakin et al., 2012; Deakin, 2014).

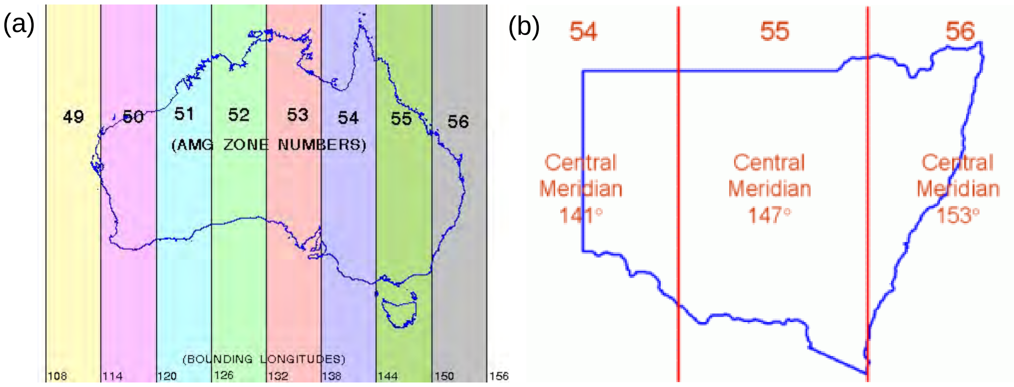


Figure 3: Map Grid of Australia (MGA) zones across (a) Australia and (b) NSW.

Transformation parameters are required to transfer data between datums. These are commonly provided by national or international agencies, generally in form of a 7-parameter or 14-parameter similarity transformation or as a transformation grid. As new datums are defined (or existing datums are refined) based on increased amounts of data and improved processing techniques, new and better transformation parameters are published. While it is acknowledged that there may be a significant delay between their initial availability and eventual adoption in software via updates or patches, it is important for users to apply the latest set of transformation parameters in order to achieve the highest possible quality of output coordinates (e.g. Dawson and Woods, 2010; Haasdyk and Janssen, 2012a).

For a review of coordinate systems, datums and associated transformations in the Australian context the reader is referred to Janssen (2009a, 2009b). A review of Australian height systems and vertical datums can be found in Featherstone and Kuhn (2006), while general background information on datums and map projections is available, for example, in Torge (2001) and Iliffe and Lott (2008).

3 DRIVERS FOR DATUM MODERNISATION

The world around us is ever-changing, so it is obvious that datum modernisation is required from time to time to benefit from technological improvements (e.g. terrestrial vs. satellite surveying techniques and advanced computing capabilities), include recent observations (e.g. larger number and increased precision), remove existing distortions, achieve a denser realisation of the datum, and provide a homogenous nationwide datum that meets the needs of today’s society.

Figure 4 illustrates the following drivers for datum modernisation in Australia:

- Including up-to-date geodetic observations and increased precision: A significant amount of additional data has been gathered since GDA94 was introduced. As an example, Figure 4a shows approximately 90,000 new GNSS baselines (blue) overlying the network of GPS observations that were used in the GDA94 adjustment in NSW (green).
- Removing known distortions: Systematic distortions of up to 0.3 m (horizontally) and ±0.3 m (vertically) have been demonstrated in GDA94 across NSW (Figure 4b). For example, currently a site transformation is required in NSW to relate CORSnet-NSW derived positions to the legal datum as realised by the Survey Control Information Management System (SCIMS – see Kinlyside, 2013) (e.g. Haasdyk et al., 2010; Janssen and McElroy, 2010; Haasdyk and Janssen, 2012b).

- Providing seamless coordinates across state borders through a nationwide simultaneous adjustment: This removes coordinate jumps at jurisdictional boundaries often introduced by running separate adjustments using different methods (Figure 4c).
- Accounting for tectonic plate motion since 1994: The Australian tectonic plate is moving at up to 7 cm/yr (~6 cm/yr in NSW) and has moved about 1.5 m north-east since 1994. By 2020, it will have moved by approximately 1.8 m (Figure 4d).
- Accounting for tectonic plate rotation: If ignored, errors of up to 7 mm are introduced for baseline lengths of only 30 km over a 20-year period (Stanaway et al., 2012) (Figure 4e).
- Introducing a truly 3-dimensional datum by appropriately considering ellipsoidal heights in the definition: This is crucial in order to realise the benefits of precise satellite-based positioning across the nation and unlock the potential for GNSS heighting (Figure 4f).

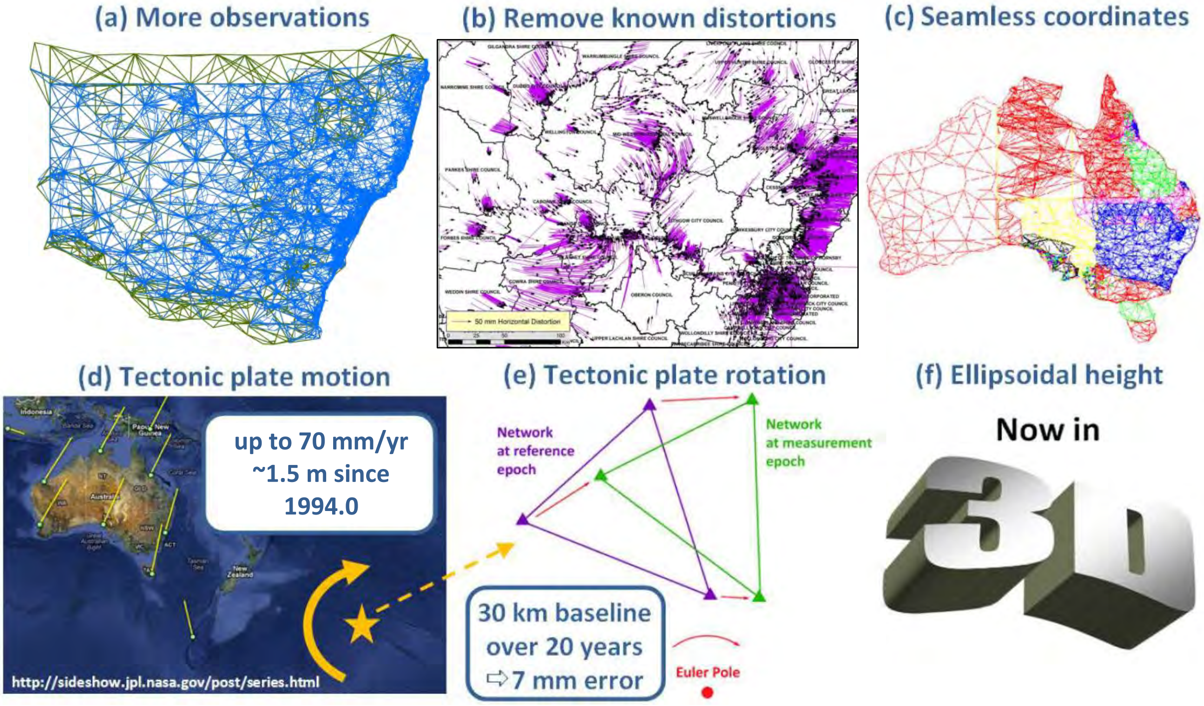
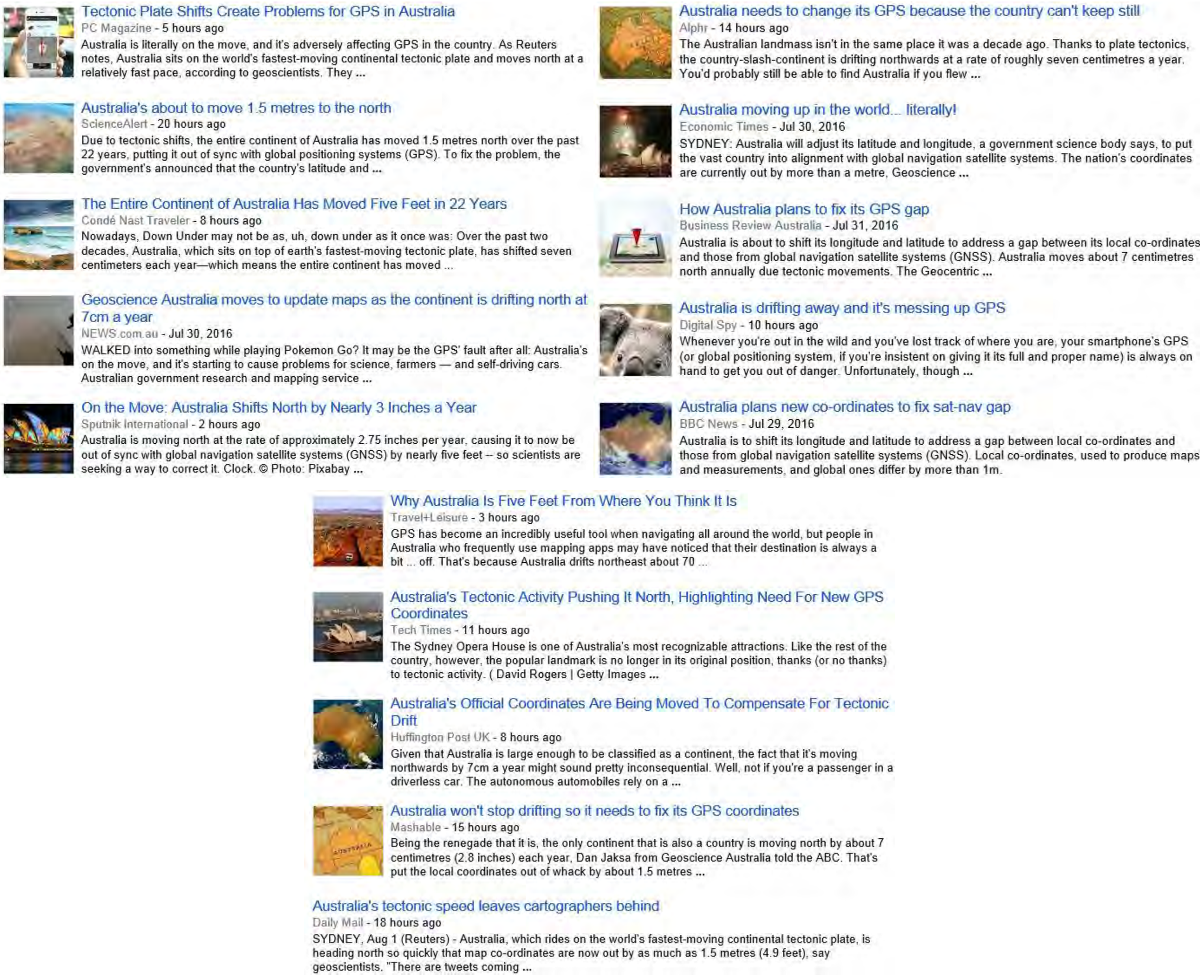


Figure 4: Some drivers for datum modernisation in Australia (adapted from Haasdyk and Watson, 2013).

In mid-2016, the plan to modernise Australia's national datum was embraced by the mainstream media, creating an international media storm about Australia moving to GDA2020 (Figure 5). Of particular note is the BBC's graphical representation of the difference between GDA94 and the current position of the Australian plate (Figure 6). The surveying and spatial information community was quite surprised to see such media hype related to a well-known and understood phenomenon, but also well aware of the sensationalism employed in the mainstream media (e.g. Wallace, 2016a, 2016b).

While unexpected, this media coverage shows that precise satellite-based positioning is set to become commonplace in the mass market. Soon any mobile device, such as a smartphone or iPad, will be able to provide positioning to the layperson at the sub-metre level. These mass-market applications will utilise a global time-dependent reference frame, so it is imperative that Australia's national datum can cater for these future needs.



Tectonic Plate Shifts Create Problems for GPS in Australia
PC Magazine - 5 hours ago
Australia is literally on the move, and it's adversely affecting GPS in the country. As Reuters notes, Australia sits on the world's fastest-moving continental tectonic plate and moves north at a relatively fast pace, according to geoscientists. They ...

Australia's about to move 1.5 metres to the north
ScienceAlert - 20 hours ago
Due to tectonic shifts, the entire continent of Australia has moved 1.5 metres north over the past 22 years, putting it out of sync with global positioning systems (GPS). To fix the problem, the government's announced that the country's latitude and ...

The Entire Continent of Australia Has Moved Five Feet in 22 Years
Condé Nast Traveler - 8 hours ago
Nowadays, Down Under may not be as, uh, down under as it once was: Over the past two decades, Australia, which sits on top of earth's fastest-moving tectonic plate, has shifted seven centimeters each year—which means the entire continent has moved ...

Geoscience Australia moves to update maps as the continent is drifting north at 7cm a year
NEWS.com.au - Jul 30, 2016
WALKED into something while playing Pokemon Go? It may be the GPS' fault after all: Australia's on the move, and it's starting to cause problems for science, farmers — and self-driving cars. Australian government research and mapping service ...

On the Move: Australia Shifts North by Nearly 3 Inches a Year
Sputnik International - 2 hours ago
Australia is moving north at the rate of approximately 2.75 inches per year, causing it to now be out of sync with global navigation satellite systems (GNSS) by nearly five feet – so scientists are seeking a way to correct it. Clock. © Photo: Pixabay ...

Australia needs to change its GPS because the country can't keep still
Alphr - 14 hours ago
The Australian landmass isn't in the same place it was a decade ago. Thanks to plate tectonics, the country-slash-continent is drifting northwards at a rate of roughly seven centimetres a year. You'd probably still be able to find Australia if you flew ...

Australia moving up in the world... literally!
Economic Times - Jul 30, 2016
SYDNEY: Australia will adjust its latitude and longitude, a government science body says, to put the vast country into alignment with global navigation satellite systems. The nation's coordinates are currently out by more than a metre, Geoscience ...

How Australia plans to fix its GPS gap
Business Review Australia - Jul 31, 2016
Australia is about to shift its longitude and latitude to address a gap between its local co-ordinates and those from global navigation satellite systems (GNSS). Australia moves about 7 centimetres north annually due tectonic movements. The Geocentric ...

Australia is drifting away and it's messing up GPS
Digital Spy - 10 hours ago
Whenever you're out in the wild and you've lost track of where you are, your smartphone's GPS (or global positioning system, if you're insistent on giving it its full and proper name) is always on hand to get you out of danger. Unfortunately, though ...

Australia plans new co-ordinates to fix sat-nav gap
BBC News - Jul 29, 2016
Australia is to shift its longitude and latitude to address a gap between local co-ordinates and those from global navigation satellite systems (GNSS). Local co-ordinates, used to produce maps and measurements, and global ones differ by more than 1m.

Why Australia Is Five Feet From Where You Think It Is
Travel+Leisure - 3 hours ago
GPS has become an incredibly useful tool when navigating all around the world, but people in Australia who frequently use mapping apps may have noticed that their destination is always a bit ... off. That's because Australia drifts northeast about 70 ...

Australia's Tectonic Activity Pushing It North, Highlighting Need For New GPS Coordinates
Tech Times - 11 hours ago
The Sydney Opera House is one of Australia's most recognizable attractions. Like the rest of the country, however, the popular landmark is no longer in its original position, thanks (or no thanks) to tectonic activity. (David Rogers | Getty Images ...

Australia's Official Coordinates Are Being Moved To Compensate For Tectonic Drift
Huffington Post UK - 8 hours ago
Given that Australia is large enough to be classified as a continent, the fact that it's moving northwards by 7cm a year might sound pretty inconsequential. Well, not if you're a passenger in a driverless car. The autonomous automobiles rely on a ...

Australia won't stop drifting so it needs to fix its GPS coordinates
Mashable - 15 hours ago
Being the renegade that it is, the only continent that is also a country is moving north by about 7 centimetres (2.8 inches) each year, Dan Jaksa from Geoscience Australia told the ABC. That's put the local coordinates out of whack by about 1.5 metres ...

Australia's tectonic speed leaves cartographers behind
Daily Mail - 18 hours ago
SYDNEY, Aug 1 (Reuters) - Australia, which rides on the world's fastest-moving continental tectonic plate, is heading north so quickly that map co-ordinates are now out by as much as 1.5 metres (4.9 feet), say geoscientists. "There are tweets coming ...

Figure 5: Snapshot of worldwide online media coverage taken on 2 August 2016.

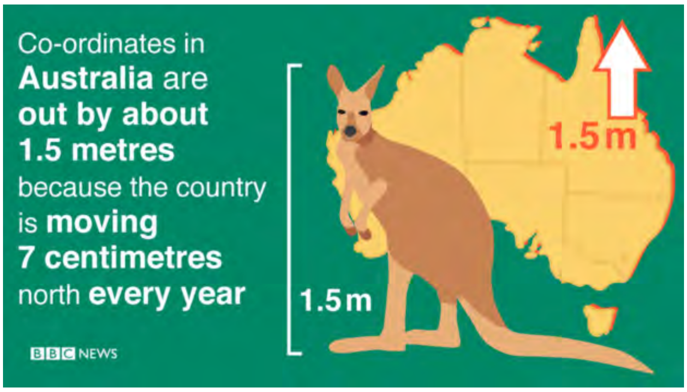


Figure 6: Mainstream media representation of Australia's tectonic motion since 1994 (Foxx, 2016).

4 PLATE-FIXED GEOCENTRIC DATUMS IN AUSTRALIA

A plate-fixed datum is attached to the tectonic plate and therefore also known as a static datum. It is 'frozen' at a certain instant in time (the reference epoch), essentially preventing the coordinates from changing over time due to tectonic plate motion. A geocentric datum uses the earth's centre of mass as its origin and is therefore compatible with GNSS-based positioning. However, as the time difference between the reference epoch and the current epoch increases, the plate-fixed datum deviates more and more from the earth-fixed datum

used for GNSS (see section 6). Consequently, it needs to be updated at frequent intervals in order to meet user requirements into the future.

4.1 GDA94

The Geocentric Datum of Australia 1994 (GDA94) has been our national datum since its adoption on 1 January 2000, providing fundamental positioning infrastructure for Australia (ICSM, 2014). The resulting coordinates are based on the Geodetic Reference System 1980 (GRS80) ellipsoid, a geocentric ellipsoid designed to approximate the earth on a global scale.

GDA94 was defined in the then state-of-the-art global reference frame, the International Terrestrial Reference Frame 1992 (ITRF92) at epoch 1994.0 (see section 6.1), realised by the eight Australian Fiducial Network (AFN) sites, and has since been 'frozen' in a geodetic sense in order to avoid changing coordinate values. This definition was justified by the relatively uniform drift of the Australian continent at ~7 cm/yr to the north-east. However, tectonic plate motion causes the difference between absolute ITRF coordinates and GDA94 coordinates to increase over time, amounting to about 1.5 m at present. This is generally not an issue for differential GNSS applications within Australia, as both ends of a baseline move at the same rate if we ignore rotation. However, the ever-increasing number of mass-market applications routinely operates in the ITRF, causing this offset to be a confusing annoyance for the layperson. While GDA94 was recently re-gazetted with improved accuracy for 21 AFN sites, it remains only indirectly linked to the current global reference frame via ITRF92 (ICSM, 2014).

4.2 GDA2020

The Geocentric Datum of Australia 2020 (GDA2020) is a new and more homogeneous conventional static datum, fixed to the Australian plate. The resulting coordinates continue to be based on the GRS80 ellipsoid. While GDA94 is constrained to a small set of static gazetted coordinates, GDA2020 is constrained to the best-available ITRF coordinates for a subset of GNSS Continuously Operating Reference Stations (CORS) contributing to the Asia-Pacific Reference Frame (APREF – see GA, 2017a) – the Australian Regional GNSS Network (ARGN) and Tier 2 AuScope sites (GA, 2017c). Consequently, for GDA2020, the coordinates of about 150 GNSS CORS are to be gazetted.

GDA2020 is defined in the global ITRF2014 frame (see section 6.1) at epoch 2020.0, i.e. the coordinates are extrapolated into the future to 1 January 2020 in order to extend the lifespan of the datum. GDA2020 is expected to be released soon, along with associated products such as GDA94-GDA2020 transformation parameters and AUSGeoid2020 (see section 5). AUSPOS, Geoscience Australia's free online GPS processing service, will provide solutions in GDA94, GDA2020 and ITRF2014 (GA, 2017b).

Following a transition period, it is envisioned that GDA2020 will be adopted by all users by 1 January 2020, although several jurisdictions may decide to move to the new datum earlier. The move from GDA94 to GDA2020 will cause the horizontal coordinates of a mark to shift by approximately 1.8 m to the north-east, while the ellipsoidal height will decrease by about 0.1 m. In order to connect to the Australian Height Datum, it is therefore crucial to apply AUSGeoid2020 to GDA2020 ellipsoidal heights, while AUSGeoid09 must be used to convert GDA94 ellipsoidal heights. Under *no* circumstance should a user combine GDA2020 with AUSGeoid09 or GDA94 with AUSGeoid2020.

The Universal Transverse Mercator (UTM) projection (e.g. Janssen, 2009a, 2009b) will continue to be used to project latitude and longitude to grid coordinates (Easting, Northing, Zone). In regards to GDA2020, these grid coordinates will be expressed in the Map Grid of Australia 2020 (MGA2020).

It is important to note that differences of up to several centimetres in both horizontal and vertical coordinates can result from following different transformation paths between two datums (Haasdyk and Janssen, 2012a). The highest and most consistent coordinate quality is obtained by following the most direct transformation path and applying the latest transformation parameters to the original, raw data. Geoscience Australia will publish a paper detailing the transformation parameters between GDA94, GDA2020 and ITRF2014. The transformation between GDA94 and GDA2020 will include a distortion grid option that accounts for the significant distortions inherent in GDA94 – this is particularly important for users in NSW. This distortion grid will also account for localised deformation, e.g. due to subsidence caused by mining or groundwater extraction.

It is anticipated that the national GDA2020 adjustment will be re-run periodically to improve the datum by incorporating new measurements, fixing problem areas and possibly retiring older, superseded measurements.

4.3 NSW's Contribution to GDA2020

CORSnet-NSW is Australia's largest state-owned and operated network of permanent GNSS reference stations. It is built, owned and operated by Spatial Services, a unit of the NSW Department of Finance, Services and Innovation (Janssen et al., 2016; DFSI Spatial Services, 2017). As of February 2017, the network consists of 189 reference stations, providing fundamental positioning infrastructure that is accurate, reliable and easy-to-use for a wide range of users (Figure 7).

This network also provides the backbone for GDA2020 across NSW. The GNSS baselines observed as part of the CORSnet-NSW local tie surveys (Gowans and Grinter, 2013) are crucial to link the GDA2020 coordinates of the CORS to the existing survey ground control network in the national adjustment. This provides a homogeneous national datum realisation across NSW, thereby significantly improving the State's geodetic infrastructure for years to come.

In support of these efforts, DFSI Spatial Services has performed extensive data-mining and cleaning of archived GNSS and terrestrial observations (e.g. Haasdyk and Watson, 2013; Haasdyk et al., 2014a) and conducted targeted state-wide new GNSS observing campaigns (e.g. Gowans et al., 2015).

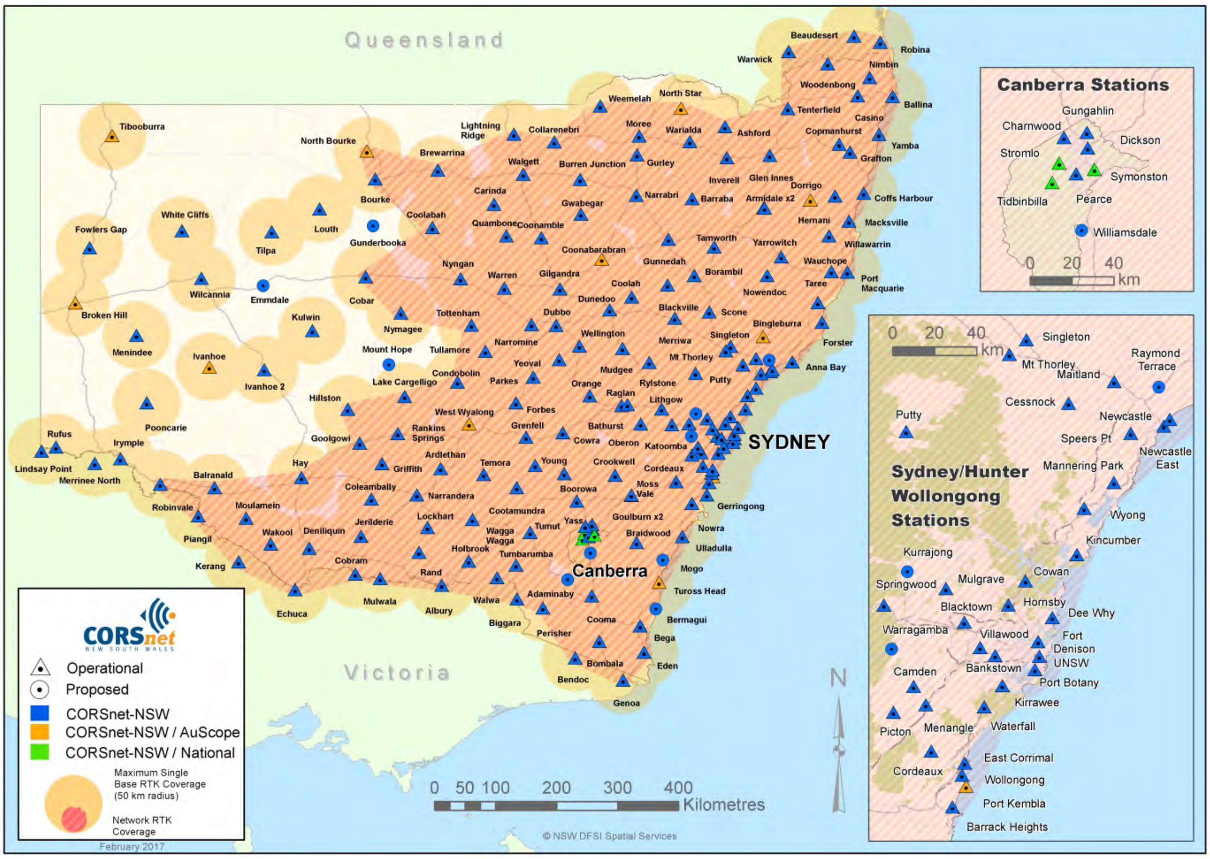


Figure 7: CORSnet-NSW network map as of February 2017 (DFSI Spatial Services, 2017).

NSW's contribution to GDA2020 version 1.0 was delivered to Geoscience Australia on 30 September 2016. The current contribution (version 1.2), incorporating additional data, was delivered to Geoscience Australia on 6 February 2017 (Gowans, 2017). It included 31,000 stations, 96,000 GNSS baselines, 5,000 GNSS baseline clusters (i.e. 6+ hour long AUSPOS sessions), 26,000 directions and 4,600 distances (Figures 8 & 9). While Figure 8 appears very busy, it clearly illustrates the vast amount of data involved in this adjustment: GNSS baseline cluster (National GNSS Campaign Archive, NGCA – purple), GNSS baselines (blue), directions (red), MSL/ellipsoidal distances (dark red), point cluster (APREF – large red circles) and all nodes (any point in the adjustment – small red circles).

This dataset was combined with the contributions from the other States and Territories to produce, for the first time, a nationwide simultaneous network adjustment for Australia. In fact, Australia is the first country in the world to attempt a continental network adjustment of this size in a single step that rigorously propagates uncertainty. The results were also used to generate the first version of official GDA2020 products, such as AUSGeoid2020 and the GDA94-GDA2020 transformation parameters and distortion grid, and then interrogated, analysed and validated by experts around the country before their initial publication. For more details about the NSW contribution, the reader is referred to Gowans (2017).

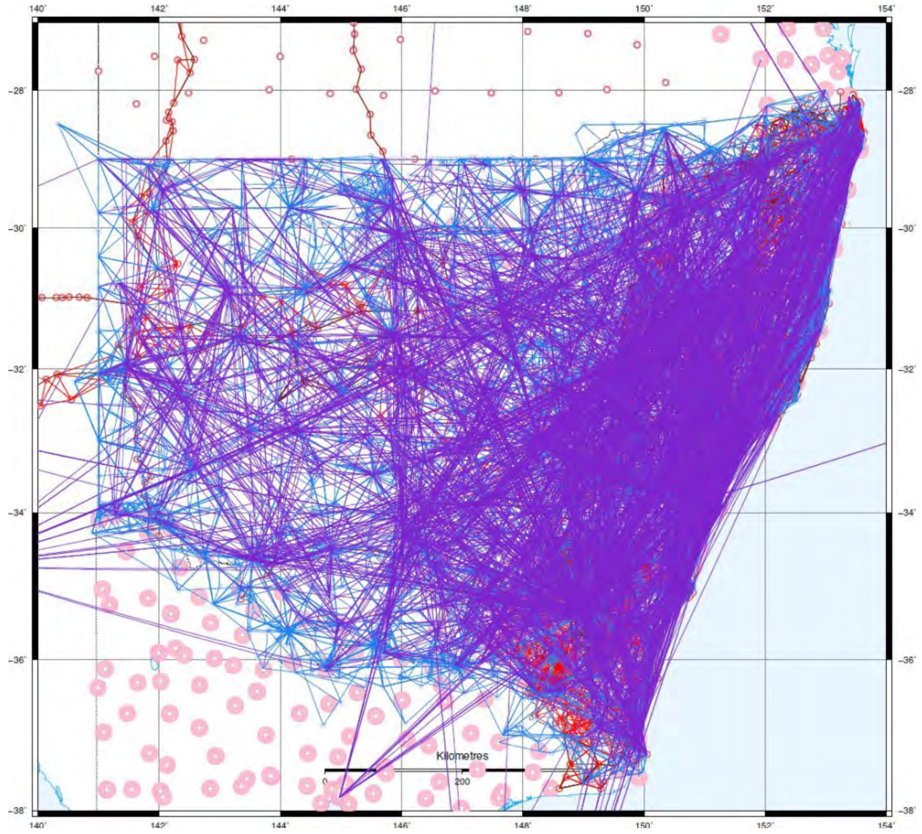


Figure 8: NSW's contribution to GDA2020 version 1.2, submitted on 6 February 2017.

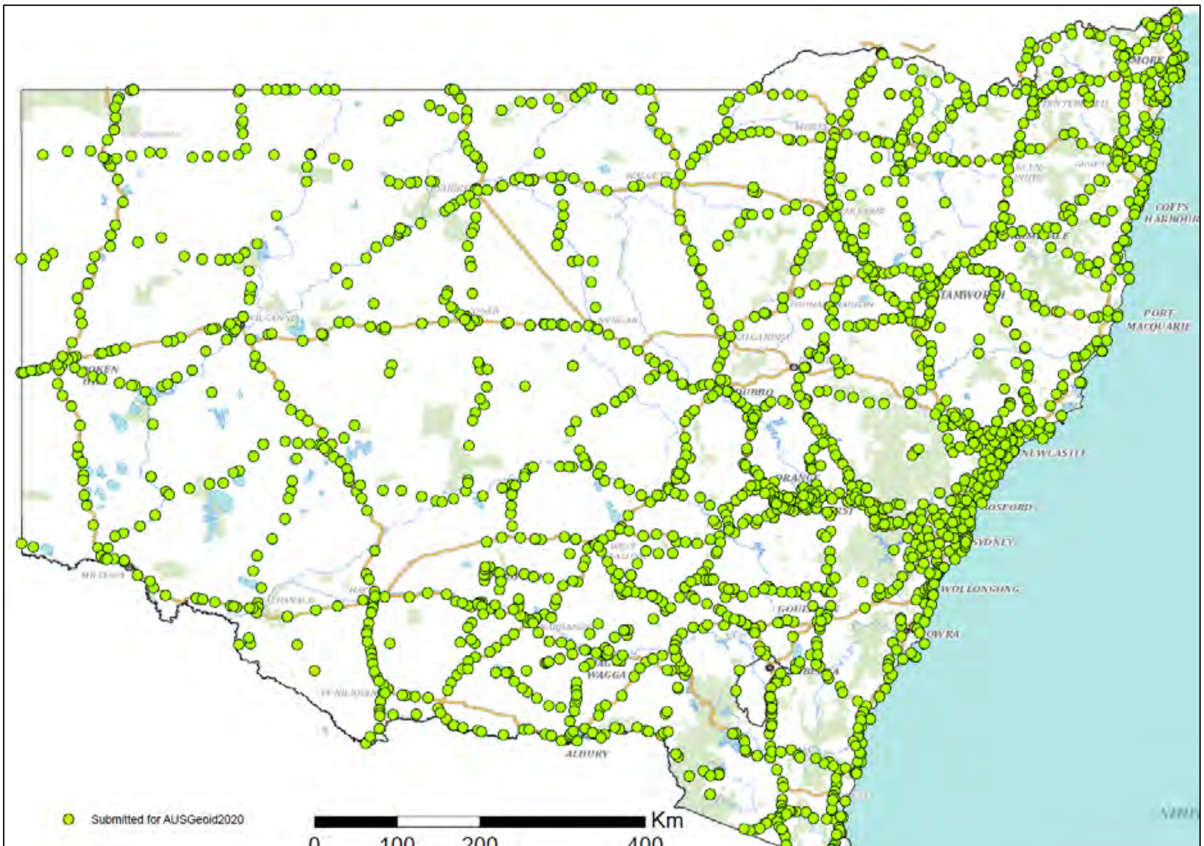


Figure 9: AUSPOS sessions (6+ hours) collected on established level marks in NSW and contributing to AUSGeoid2020 (as of February 2017).

5 AUSGeoid2020

Vertical coordinates continue to be referred to the Australian Height Datum (AHD) (Roelse et al., 1975). It is well known that shortcomings in the AHD realisation (AHD71 for mainland Australia and AHD83 for Tasmania) resulted in considerable distortions of up to about 1.5 m into AHD across Australia, which is therefore considered a third-order datum (e.g. Morgan, 1992; Featherstone and Filmer, 2012; Watkins et al., 2017). However, in the immediate future AHD continues to be a practical height datum that provides a sufficient approximation of the geoid for many surveying and engineering applications. In the longer term, the Intergovernmental Committee on Surveying and Mapping (ICSM) will consider updating AHD or replacing it potentially with a new national gravity-based vertical reference frame.

The growing use of CORS networks for GNSS-based height transfer has substantially increased the importance of accurate, absolute N values (or geoid undulations). These N values (N) can be used to convert ellipsoidal heights (h) to AHD heights (H) and vice versa, provided N and h refer to the same ellipsoid:

$$H = h - N \tag{1}$$

Fortunately, the AUSGeoid09 model has been shown to provide N values with unprecedented absolute accuracy across NSW and Australia (e.g. Janssen and Watson, 2010, 2011; Brown et al., 2011; Allerton et al., 2015; Sussanna et al., 2017).

As outlined in section 4.2, the move from GDA94 to GDA2020 causes ellipsoidal heights to decrease by about 0.1 m. Consequently, a new AUSGeoid model is necessary to connect GDA2020 ellipsoidal heights to AHD. AUSGeoid2020 is expected to be released soon together with GDA2020. While AUSGeoid2020 has the same extent (between 108ºE and 160ºE longitude and between 8ºS and 46ºS latitude) and density (1’ by 1’ grid, i.e. approximately 1.8 by 1.8 km) as its predecessor AUSGeoid09, it is based on a much larger and much more homogeneous dataset and expected to provide a significant improvement for GNSS-based heighting across the nation. However, it is crucial that AUSGeoid2020 is *only* used in conjunction with GDA2020 ellipsoidal heights. Users wishing to convert GDA94 ellipsoidal heights to AHD *must* continue to use AUSGeoid09.

Over the last few years, DFSI Spatial Services has conducted several targeted GNSS observing campaigns (e.g. ‘Saving AHD’ and ‘Positioning Rural NSW’) to collect long-duration GNSS datasets on levelled benchmarks in order to improve the AUSGeoid product across the State (see Figure 9). These efforts continue to date in order to maintain and upgrade the State’s fundamental levelling infrastructure and increase the density of levelled benchmarks occupied with GNSS across the State. So far, some 600-800 original 1970s Australian National Levelling Network (ANLN) marks (i.e. typically about 6 marks per level run in rural areas) were preserved and upgraded to meet current and future requirements. In addition, long GNSS sessions have been observed at more than 70 existing LAL1 (first order) and LBL2 (second order) levelling marks, covering the area from north of Newcastle to south of Wollongong at an average density of about one mark every 10 km.

6 EARTH-FIXED GEOCENTRIC DATUMS IN AUSTRALIA

An earth-fixed datum accounts for the earth's dynamics by allowing tectonic plates to move within it. It is fixed to the earth but not its crust and therefore also known as a dynamic datum. Consequently, the coordinates of a given ground mark are constantly changing. Hence it is critical to attach a time stamp to each position given in an earth-fixed datum, so a position given at a reference epoch can be propagated to the current or any other epoch using station velocities. The reference epoch represents a date and time that is conveniently agreed upon to assist with the meaningful transfer of coordinates, measurements and other parameters.

6.1 ITRF

The International Terrestrial Reference Frame (ITRF) is the most precise earth-centred, earth-fixed datum currently available and was first introduced in 1988. It is maintained by the International Earth Rotation and Reference Systems Service (IERS) and realised by an extensive global network of accurate coordinates and their velocities derived from geodetic observations using GNSS, Very Long Baseline Interferometry (VLBI), Satellite Laser Ranging (SLR) and Doppler Orbitography and Radiopositioning Integrated by Satellite (DORIS) (Altamimi et al., 2016).

The ITRF is a *time-dependent* (or dynamic) datum and changes according to temporal variations of its network coordinates and their velocities due to the effects of crustal motion, earth orientation, polar motion and other geophysical phenomena such as earthquakes and volcanic activity (Bock, 1998). It is updated regularly in order to account for the dynamics of the earth and now sufficiently refined to ensure that the change between successive ITRF versions is in the order of a few millimetres. So far the following 13 versions have been released: ITRF88, ITRF89, ITRF90, ITRF91, ITRF92, ITRF93, ITRF94, ITRF96, ITRF97, ITRF2000, ITRF2005, ITRF2008 and ITRF2014.

Figure 10 illustrates the ITRF2014 network, consisting of 1,499 stations located at 975 sites, with about 10% collocated with up to four distinct space geodetic instruments. The resulting coordinates are based on the GRS80, a geocentric ellipsoid designed to approximate the earth on a global scale. It is worth noting that the current realisation (ITRF2014 with reference epoch 2010.0) incorporates, for the first time, the non-linear motion of the earth's surface caused by post-seismic relaxation after earthquakes, resulting in a significant improvement. The origin shift from ITRF2008 to ITRF2014 amounts to a mere 3.5 mm (Altamimi et al., 2016).

Coordinates given in any of the ITRF realisations are referred to a specific *epoch* in order to enable appropriate consideration of the earth's dynamics. Unlike GDA94 and GDA2020, which are fixed to the Australian plate and therefore ignore crustal motion, point coordinates given in the ITRF change over time as the tectonic plate they sit on is moving. As a result, it is important to time-stamp each set of coordinates in order to specify which epoch the position refers to. When combining data observed at different points in time (e.g. during a longer-term infrastructure project), all data have to be propagated to the same (arbitrary) epoch to enable comparison. This can be achieved, for example, via utilising a global plate motion model or a local deformation model based on GNSS CORS time series.

The epoch should be declared in decimal years, e.g. ITRF2014(2017.218) indicates a positon in ITRF2014 valid at 12:00 UT on 21 March 2017. The decimal is calculated by day of year

(80) minus one, plus time in the day (0.5 days), divided by the number of days in the year (365, considering that 2017 is not a leap year).

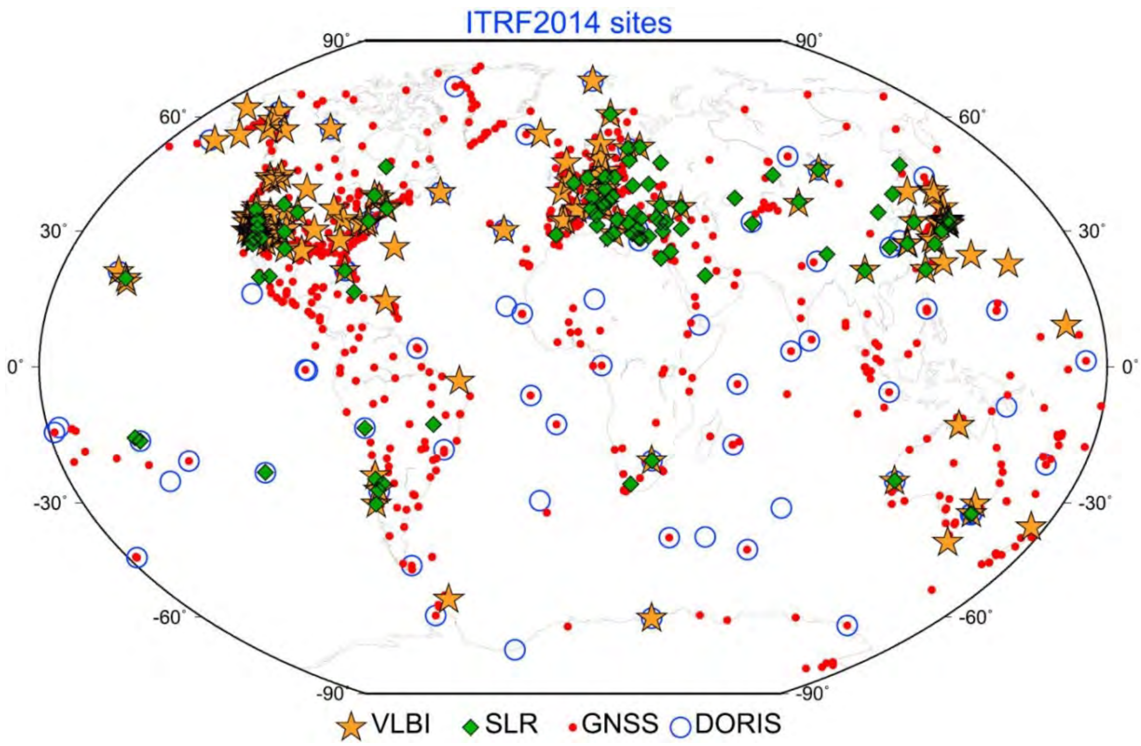


Figure 10: ITRF2014 network, highlighting VLBI, SLR and DORIS sites collocated with GNSS (Altamimi et al., 2016).

6.2 ATRF

The Australian Terrestrial Reference Frame (ATRF) is basically a regional realisation of the ITRF, based on almost 200 globally distributed International GNSS Service (IGS) stations and the Asia-Pacific Reference Frame (APREF) network consisting of about 500 stations (GA, 2017a). As a consequence, Australian spatial information will be directly interoperable with GNSS measurements, i.e. mass market applications will not see a discrepancy between global coordinates and ATRF.

It is anticipated that the ATRF will be implemented from January 2020 with adoption planned to be complete by 2023. However, it is important to note that GDA2020 and ATRF will exist in tandem for the foreseeable future. Practically, GDA2020 and ATRF can really be thought of as one product – the former being fixed to epoch 2020.0, while the latter refers to the current epoch (or any other user-specified epoch).

6.3 WGS84

The World Geodetic System 1984 (WGS84) was developed for the U.S. Defense Mapping Agency (DMA), later named NIMA (National Imagery and Mapping Agency) and now called NGA (National Geospatial-Intelligence Agency), and is the nominal datum used by GPS (NIMA, 2004). It is based on the WGS84 ellipsoid which can generally be assumed identical to the GRS80. The WGS84 datum was introduced in 1987 and has since been refined several times to be closely aligned with the ITRF in order to prevent degradation of the GPS broadcast ephemerides (i.e. orbit parameters) due to plate tectonics. The following

refinements have been released to date: WGS84(G730), WGS84(G873), WGS84(G1150) and WGS84(G1674) (Donnelly et al., 2014). In this context, G1674 denotes the GPS week in which the latest realisation of WGS84 was implemented. For all mapping and charting purposes, WGS84 and the most current ITRF can be assumed identical. However, it should be noted that WGS84 is based on a much smaller number of globally distributed reference stations than the ITRF, and the level of agreement worsens as the time gap between WGS84 and the latest realisation of ITRF grows.

It is also worth noting that, in single point positioning mode, the datum of the satellite orbits determines the datum of the resulting coordinates. Consequently, in practice, a user's stand-alone *absolute* GNSS position is nominally in WGS84 if broadcast orbits are used and nominally in ITRF if precise orbits are used (e.g. via online Precise Point Positioning services). A transformation can then be applied to refer the position to a national datum. Users connecting to the Australian control network (such as SCIMS in NSW) or an Australian CORS network (such as CORSnet-NSW) via *relative* measurements will, of course, obtain positions in Australia's national datum. While relative measurements often utilise broadcast orbits during processing, these are not used to determine the datum of the coordinates being generated.

7 TRANSITION TO GDA2020 AND ATRF

The ICSM's GDA Modernisation Implementation Working Group (GMIWG) is responsible for assisting with the transition to GDA2020 and ATRF (ICSM, 2017). This is achieved through stakeholder engagement and the development of tools, technical resources and educational material in order to facilitate a smooth transition with minimal disruption to existing systems and processes (Jaksa, 2015; ICSM, 2016b).

It is anticipated that the transition from GDA94 to GDA2020 will be complete by 1 January 2020, i.e. the date ATRF is planned to be released. However, most State and Territories are expected to move to GDA2020 before this date. The GDA94-GDA2020 transformation parameters and the new AUSGeoid2020 model, along with relevant information on their correct use, will soon be available from ICSM and Geoscience Australia.

In the context of datum modernisation, it is crucial to stress the importance of metadata. Unfortunately, metadata management is often not approached with appropriate rigour and datasets are often manipulated without keeping the metadata records up to date. However, considering the transition period between GDA94 and GDA2020 (and then towards ATRF), it is obvious that metadata is becoming just as important as the datasets themselves. Appropriate metadata records are essential to enable datasets to be traced back to their source, thereby allowing improved models to be applied in the future. All datasets should include complete metadata about the epoch, datum, method of collection and estimated uncertainty. Ideally, the original data should be archived unchanged at the observation epoch (or at least by recording any transformations or alterations applied to the raw data) in order to allow reprocessing of the original data at a later stage.

Looking further ahead, it is important to note that GDA2020 and ATRF will operate together as a dual frame system for the foreseeable future. This means that generally it will be up to the user to decide whether they would like to adopt GDA2020 or ATRF as the datum for their data holdings or a particular job, although some spatial information will be legally mandated

in GDA2020 and eventually ATRF. GDA94 will continue to exist in a legal and practical sense for some time, but it is anticipated that the requirement for higher precision and accuracy will strongly drive the move to GDA2020 and ATRF.

8 CONCLUDING REMARKS

On behalf of the Surveyor General, DFSI Spatial Services has a legislative, regulative responsibility to maintain the geodetic control network in NSW. As such, DFSI Spatial Services is the custodian of some 250,000 marks in SCIMS, which includes about 6,000 traditional 'passive' trigonometrical stations as well as about 190 'active' GNSS CORS belonging to CORSnet-NSW. Keeping the geodetic component of the survey control network current and ready for utilisation requires regular maintenance and upgrade. Australian datum modernisation will provide significant improvements for all users of spatial information across the State and the nation.

The new Australian datum, GDA2020, is expected to be released soon, along with associated products such as GDA94-GDA2020 transformation parameters and AUSGeoid2020. It is anticipated that it will be adopted by most jurisdictions before the target date 1 January 2020. Consequently, users need to be aware of the change and its importance in regards to making the most of the improved geodetic infrastructure by relating their datasets to the new datum.

This paper has provided some background information on coordinate systems, datums and map projections before briefly exploring the need for datum modernisation in Australia. It has explained the terms GDA2020, AUSGeoid2020 and ATRF (along with related terms) in general and related them to the future move from a plate-fixed datum (GDA2020) to an earth-fixed datum (ATRF). The considerable efforts undertaken at DFSI Spatial Services to support datum modernisation in NSW have also been summarised.

The ongoing datum modernisation will provide a much improved Australian national datum that will ensure that Australia is well positioned into the future.

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GDA2020 in NSW

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ABSTRACT

The first public release of the Geocentric Datum of Australia 2020 (GDA2020) has been computed along with its associated products: AUSGeoid2020, transformation parameters, and distortion grid. For several years, NSW has been building and expanding CORSnet-NSW into Australia's largest operator-owned Global Navigation Satellite System (GNSS) Continuously Operating Reference Station (CORS) network, conducting targeted state-wide new GNSS observing campaigns, and collating decades of existing geodetic measurements to contribute to this new national survey adjustment. This paper provides a brief overview of GDA2020 while detailing NSW's efforts and contribution to date. The performance of GDA2020 across NSW is evaluated in terms of positional uncertainty, and using a series of GDA94-GDA2020 case studies.

KEYWORDS: *GDA94, GDA2020, APREF, datum modernisation.*

1 INTRODUCTION

The first public release of the Geocentric Datum of Australia 2020 (GDA2020) has been produced by Geoscience Australia, with contributions from all Australian states and territories as part of Stage 1 of the datum modernisation effort (ICSM, 2017a). GDA2020 is a new, plate-fixed datum, aligned to the International Terrestrial Reference Frame 2014 (ITRF2014 – see Altamimi et al., 2016) and modelled forward to a reference epoch of 2020.0.

GDA2020 provides significant improvements in datum realisation in NSW and will succeed GDA94 in Australia as the recognised value standard for position by 1 January 2020. GDA2020 is a contiguous survey adjustment constrained to state-of-the-art Global Navigation Satellite System (GNSS) Continuously Operating Reference Stations (CORS) from the Asia-Pacific Reference Frame (APREF – see GA, 2017a), and densified with a combination of geodetic-grade GNSS and terrestrial measurements. Of particular significance to NSW is that GDA2020 is a truly 3D adjustment and will allow the establishment of true ellipsoidal height across the State's survey control network. GDA2020 is the world's first continental-scale survey adjustment to rigorously propagate uncertainty.

For background information on the specifics of the national GDA2020 adjustment and its products, the reader is referred to Janssen (2017). This paper will focus on the NSW effort towards GDA2020 and provide a preliminary evaluation of its performance in NSW via a series of case studies.

2 GDA2020 IN NSW

On behalf of the Surveyor General, Spatial Services, a unit of the NSW Department of Finance, Services and Innovation (DFSI), has a legislative and regulative responsibility to establish and maintain the geodetic control network across New South Wales. As such, DFSI Spatial Services has been preparing for the GDA2020 survey adjustment for several years with efforts to source, harvest, clean and utilise legacy geodetic measurements (Haasdyk and Watson, 2013), build state-of-the-art GNSS CORS network infrastructure (CORSnet-NSW – see Janssen et al., 2016; DFSI Spatial Services, 2017), observe new high-quality GNSS measurements to connect the existing survey network to CORS (Gowans and Grinter, 2013), and systematically rationalise, maintain and upgrade key sites across the state's trigonometrical (trig) station and Australian Height Datum (AHD) levelling networks (Gowans et al., 2015).

2.1 GDA2020 Adjustment: NSW Contribution

The NSW contribution to GDA2020 currently (February 2017) consists of approximately 335,000 measurements across 31,000 stations. The type and quantity of measurements involved are listed in Table 1.

Table 1: Measurement types and quantities contributing to GDA2020 from NSW.

Measurement Type	Quantity	Present in NSW GDA94 Contribution
Geodetic azimuth	324	Yes
Directions	26,249	Yes
Ellipsoid arc	4,639	Yes
GNSS baseline component	287,511	8,742
Orthometric height	2,052	Yes
Level difference	416	Yes
GNSS baseline cluster component	15,129	nil
GNSS point cluster component	528	nil

These measurement totals are composed of a number of new and legacy survey campaigns, dating as far back as the Australian Geodetic Datum 1966 (AGD66) adjustment. Such campaigns include:

- NSW GDA94 Terrestrial Spine Network (originally in AGD66).
- NSW GDA94 GPS Spine Network.
- NSW Survey Operations GNSS adjustments.
- NSW CORSnet-NSW GNSS local tie surveys.
- Trig upgrade and maintenance campaigns.
- Saving AHD campaigns.
- Positioning Rural NSW campaigns.

Each GNSS measurement is time-stamped, allowing for, firstly, alignment to a common reference epoch to account for the tectonic movement of the Australian continental plate, and secondly, time-dependent deformation analysis to be carried out.

GDA2020 is designed to be a 'living' adjustment such that it can be readily recomputed and improved as new measurements are made available, or as blunders are detected and removed. There is a change in philosophy from storing the solution, or coordinates, to storing the measurements. It is the intent of DFSI Spatial Services to continue contributing new and additional legacy survey measurements (such as street-corner traversing networks) as they

become available to GDA2020 in an effort to continue to produce the best possible survey network adjustment for NSW.

2.2 GDA2020 Adjustment: NSW Uncertainty Results

NSW's survey network quality is currently evaluated according to hierarchical systems of Class and Order based on the superseded SP1 v1.7 (ICSM, 2007; Dickson, 2012). The GDA2020 least squares network adjustment is computed with DynaNet using a phased-adjustment least squares methodology and provides rigorous uncertainty across the entire network (Fraser et al., 2014). This result affords DFSI Spatial Services with the capability to adopt SP1 v2.1 (ICSM, 2014), which requires survey network evaluation in terms of positional (composed of both horizontal and vertical components), relative, or survey uncertainties.

An initial analysis shows that of the approximately 31,000 NSW stations included in the adjustment, more than 80% achieve a horizontal uncertainty of better than 0.02 m (Table 2). This is represented visually in Figure 1.

Table 2:GDA2020 uncertainties in NSW's survey control network.

Positional Uncertainty (95% CI)	Horizontal %	Vertical %
< 0.02 m	81.7	65.2
< 0.05 m	94.6	89.7
> 1 m	1.2	6.2

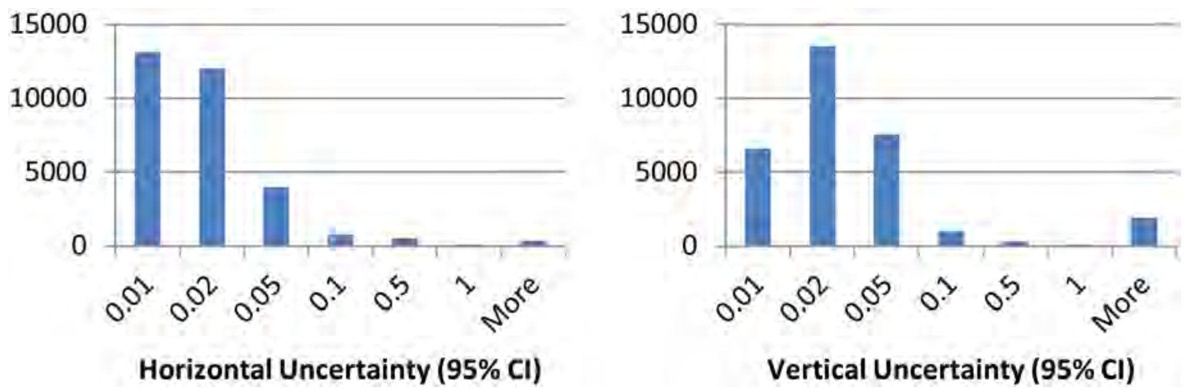

Figure 1: Horizontal and vertical uncertainty breakdowns.

The significance of this result for NSW cannot be overstated. For the first time, a state-wide survey network has been simultaneously adjusted to provide a homogeneous datum with uncertainty of better than 0.02 m at most stations. Users of the survey control network in traditional high-distortion areas of NSW will particularly benefit from the GDA2020 adjustment.

Those stations in the category greater than 1 m in horizontal uncertainty are usually part of GNSS 'island' networks, which have no connection to datum and are therefore assumed uncertainties in the order of 20 m. It is anticipated that these 'islands' will be gradually connected to datum by future targeted survey campaigns and as terrestrial adjustment data begins to be sourced, refined and contributed to the GDA2020 adjustment.

2.3 AUSGeoid2020

AUSGeoid2020 is the new national quasi-geoid for use with GDA2020. It delivers an improvement over AUSGeoid09, not only in terms of input data to the model, but in a world-first it provides uncertainty as a function of location (CRC-SI, 2017).

Like its predecessor AUSGeoid09 (Brown et al., 2011), AUSGeoid2020 is composed of both geometric and gravimetric components. The gravimetric component is based on a number of technologies and datasets such as modern earth gravity models, recent satellite-borne gravity missions, local airborne gravity datasets, and discrete absolute gravity measurements, and is therefore outside the scope of DFSI Spatial Services to contribute towards.

The geometric component of AUSGeoid2020 (the so-called 'sliver') basically fits the gravimetric quasi-geoid model to the Australian Height Datum (AHD – see Roelse et al., 1975), and it is to this component that DFSI Spatial Services has contributed over 2,500 points with co-located GNSS-AHD heights to the AUSGeoid2020 model, a remarkable improvement over the 100 co-located GNSS-AHD heights that informed AUSGeoid09 across NSW (Figure 2).

DFSI Spatial Services has made a unique and substantial investment into preservation and upgrade of its AHD survey infrastructure with its 'Saving AHD' campaigns. These measured points involve 6+ hour GNSS observations on stations which have AHD heights of class and order LCL3 or better in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013). After the ellipsoidal height (h) is determined from the GNSS measurement, the geoid undulation (N) can be computed using:

$$N_{AHD} = h_{GDA2020} - H_{AHD} \qquad (1)$$

The majority of these measured points were observed during the 'Saving AHD' campaigns (2015-16) across NSW. This unique NSW effort has been undertaken to provide a significant improvement to GNSS users' ability to deliver true local AHD. 'Saving AHD' has allowed DFSI Spatial Services to analyse and assess the accuracy of AHD71 and AUSGeoid2020 across the State, which in turn will provide for better informed decision making. At the time of writing, DFSI Spatial Services is currently evaluating the performance of AUSGeoid2020 across NSW.

It is imperative to note that AUSGeoid2020 is incompatible with GDA94 due to a change in the determination of the centre of mass of the Earth between ITRF92 and ITRF2014. Therefore any combined use of GDA94 and AUSGeoid2020, or GDA2020 and AUSGeoid09 will result in absolute heighting errors in the order of 0.1 m.

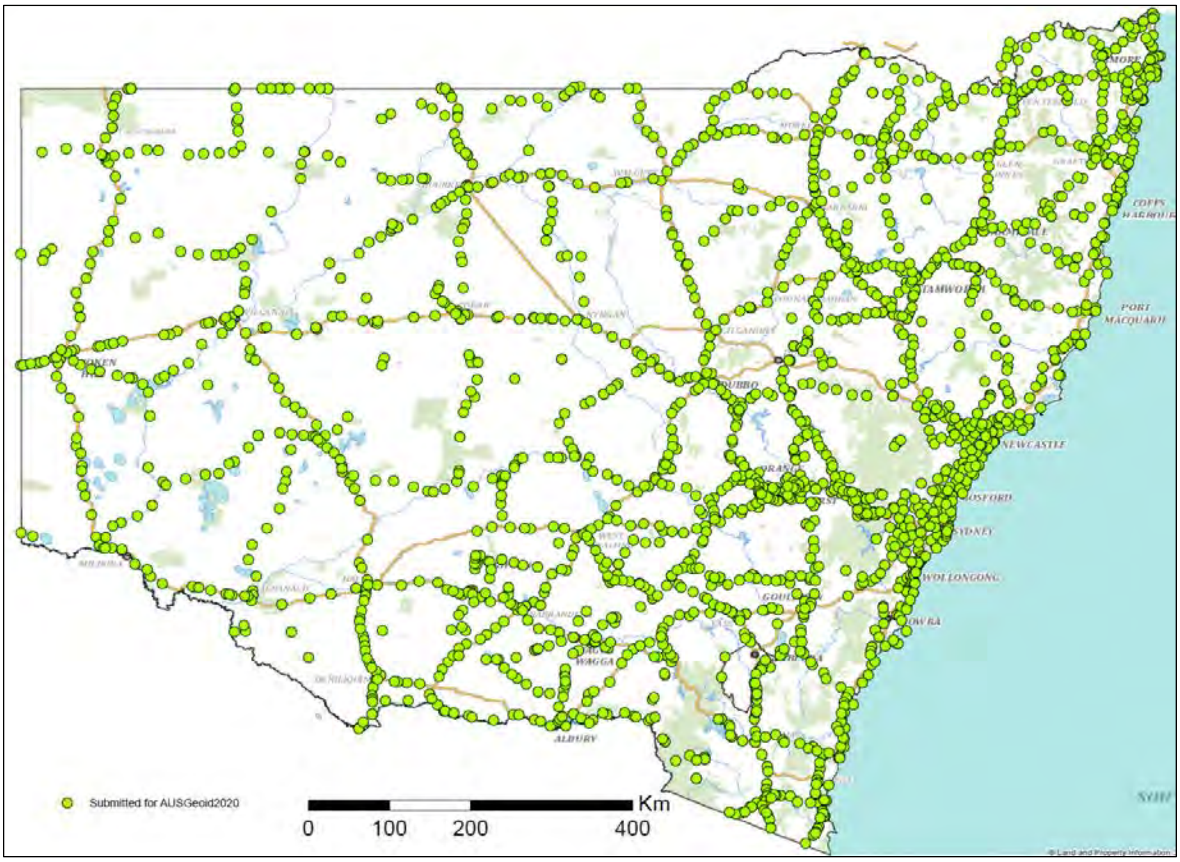

Figure 2: NSW measured points contributing to AUSGeoid2020 (as of February 2017).

2.4 GDA2020 Distortion Grid in NSW

An upcoming publication by Geoscience Australia staff will quantify the relationships between GDA94, GDA2020, and ITRF, which is required to align and meaningfully analyse spatial datasets that refer to separate reference frames. GDA94 and GDA2020 are both plate-fixed reference frames (ICSM, 2017b; Janssen, 2017) and therefore can be transformed between using a 7-parameter conformal transformation. ITRF, being a time-dependent, earth-fixed reference frame, requires known rates of change, and therefore necessitates a 14-parameter conformal transformation to relate to GDA94 or GDA2020.

It is important to note that these conformal transformations do not take into account any distortion present in the realisation of GDA94. In NSW, 2A0 Spine control has been intentionally fixed at its original realisation value (as at 1997), and this may result in errors of up to 0.3 m (e.g. Janssen and McElroy, 2010; Janssen et al., 2016).

Such stations, which may have been originally coordinated with lower-precision terrestrial measurements, were generally not updated as newer, higher-precision GNSS measurements became available. The widespread adoption of absolute GNSS positioning technologies, such as AUSPOS (GA, 2017b) or CORSnet-NSW (Janssen et al., 2016; DFSI Spatial Services, 2017), has meant that distortions are very easy to detect in areas of coarser network precision and decimetre-level differences can be found. These network distortions can complicate surveys crossing state borders, where other jurisdictions have updated coordinates as new measurements became available, as well as surveys for large infrastructure projects. For more information on dealing with GDA94 distortions in NSW, the reader is referred to Janssen and

McElroy (2010), who term the SCIMS realisation of GDA94 in NSW as GDA94(1997) and the absolute realisation offered by AUSPOS and CORSnet-NSW as GDA94(2010).

The national distortion grid will provide a transformation that accounts for such distortions. Approximately 26,000 common points between SCIMS and the GDA2020 adjustment were provided to inform the distortion grid across NSW. These points from SCIMS were selected on the basis that they are within NSW, are common between SCIMS and GDA2020, have an order of 4 or better; and produce a comparable distortion vector within the trend of the area. The distortion vectors are represented graphically in Figure 3.

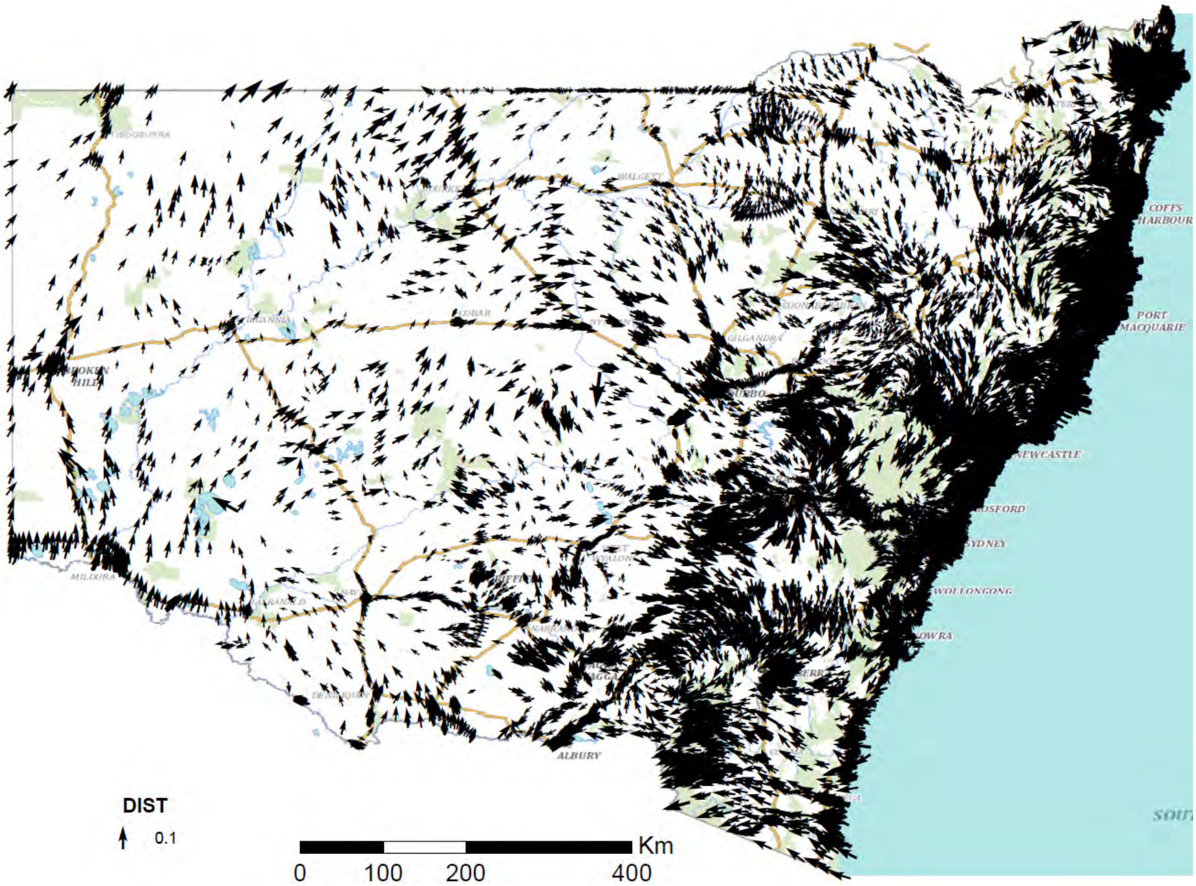

Figure 3: GDA94(1997) to GDA94(2010) distortion vectors across NSW.

3 CASE STUDIES

The case studies presented here provide an introduction into how the new Australian datum performs in NSW. For simplicity of comparison, the coordinate differences shown between original GDA94 and new GDA2020 adjustments have been corrected for the effects of tectonic motion between reference epochs, as if both datum epochs were aligned to a common reference epoch of 1 January 1994. This allows a valid comparison with current GDA94 Regulation 13 certification (GA, 2017c) to be made where appropriate.

3.1 Case Study 1: Gilgandra 2D Traverse Network

Gilgandra is a rural town in NSW’s central west. The Gilgandra traverse network consists of 296 directions and 291 distances to 99 stations. In 2013, the terrestrial network was readjusted

based on 13 control stations established in a new GNSS subspine network. The adjustment is displayed in Figure 4.

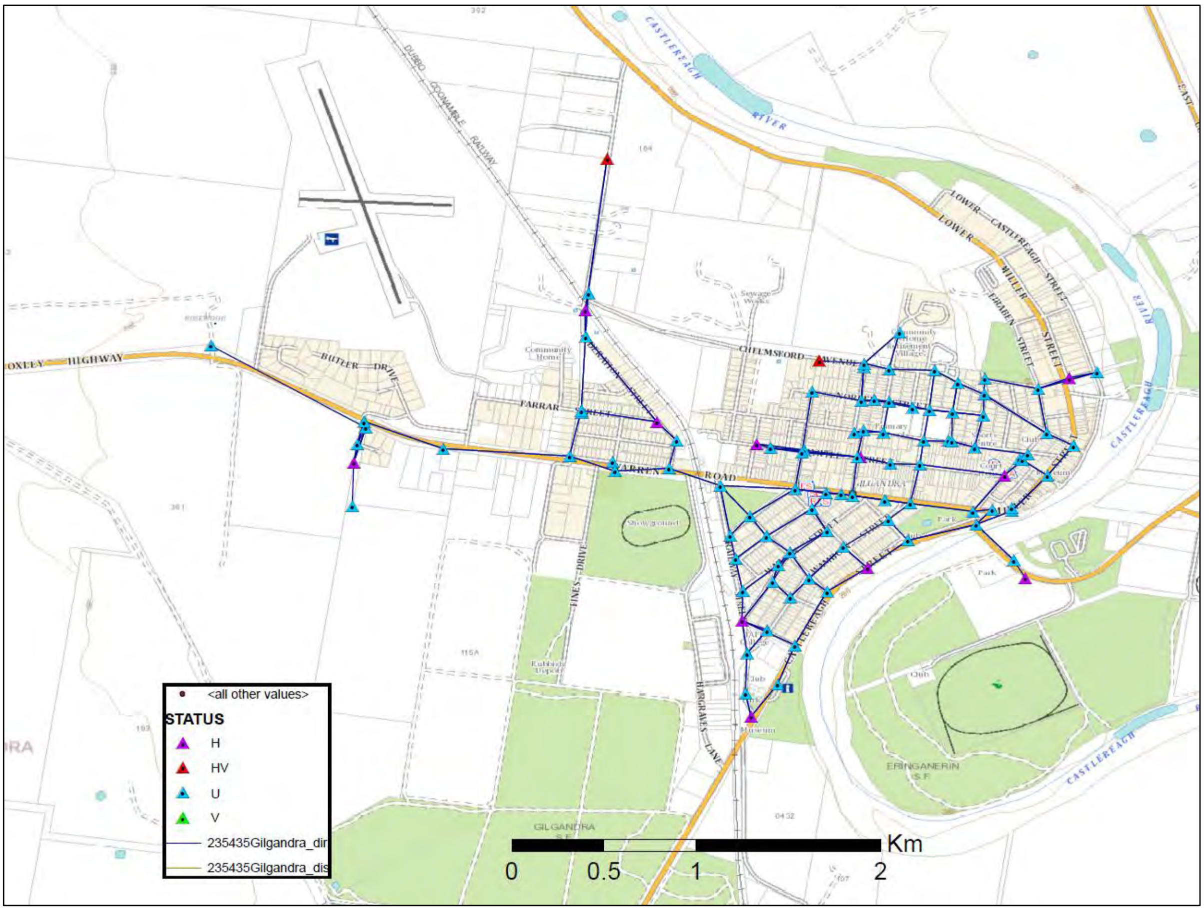

Figure 4: Gilgandra GDA94 traverse adjustment.

The current GDA94 adjustment fails the Chi-Square test on the variance factor, coming in low at 0.31, which indicates the observation standard deviations are generally more generous than their overall fit in the network. Additionally, no residuals are flagged, which indicates a clean survey network.

Re-running the adjustment in GDA2020 allows for two options. Firstly, the ‘fixed’ control coordinates can simply be updated from the result of the national adjustment, and secondly, the control can be constrained based on their station positional uncertainties as determined by the national adjustment.

Fixing the new GDA2020 control introduced two new additional control stations, for a total of 15 fixed. The adjustment still fails the Chi-Square test, with a variance factor of 0.47. No outliers are flagged, and residuals fall within their allowable tolerances. The average change to coordinates is seen to be [ΔE: +0.021 m; ΔN: -0.040 m], which is indicative of the level of distortion in GDA94(1997) in Gilgandra. Standard deviations are available on the output coordinates, but they have little relevance to their connection to datum.

Constraining the new GDA2020 control based on their uncertainties from the national adjustment results in a drop in the variance factor to 0.32. The increased number of observations, as station uncertainties, is the causative factor here. This average difference in

coordinates to the GDA94 adjustment is nearly identical at [ΔE: +0.021 m; ΔN: -0.041 m]. However, the resultant station standard deviations are now influenced by the overall network's connection to datum, and thus their 2D uncertainties can be estimated in accordance with SP1 v2.1 (ICSM, 2014). In this instance, the average 2D uncertainty is 0.013 m at the 95% confidence interval.

Overall, this study demonstrates that GDA2020 still looks and feels like GDA94, but provides significant advantages in terms of datum homogeneity and provision of rigorous uncertainty with respect to datum.

3.2 Case Study 2: Nimbin GNSS CORS Tie Survey

Nimbin CORS is located within NSW's North Coast survey network, an area notorious for its significant distortion, caused by the coarser precision of older terrestrial measurements. The CORS tie survey network was designed to connect TS12213 Nimbin CORS into the NSW survey network for the GDA2020 adjustment (Figure 5). For the purpose of this analysis, a test GDA2020 network has been created with these measurements removed to preserve the rigour of the test.

The adjustment is composed of 15 stations and 27 GNSS baselines. A high level of tension is evident in the GDA94 fixed adjustment, whose variance factor (VF) fails the Chi-Square test at VF = 4.12, having 4 outliers flagged. Residuals of up to 0.18 m, or 10 ppm, are present. In this instance, the tension is entirely caused by the measurements having a finer precision than the control available. The minimally constrained adjustment shows no issue with the measurements at VF = 0.29. In part, the low variance factor is also due to low network redundancy. In this network, the low level of redundancy is not overly concerning as the measurements are validated with the constraint of control stations.

The GDA2020 3D fixed adjustment computes with a VF of 0.58 and no flagged outliers. Coordinates for TS12213 Nimbin CORS match the Regulation 13 certificate at the centimetre level at [ΔE: +0.010 m; ΔN: +0.008 m; ΔEHGT: -0.013 m]. This shows that GNSS CORS and the passive ground survey stations are now aligned to within GNSS measurement precisions. The average change to horizontal coordinates is seen to be [ΔE: -0.090 m; ΔN: +0.082 m] which is indicative of the level of distortion in GDA94(1997) around Nimbin.

The GDA2020 3D constrained adjustment fails the Chi-Square test with a VF of 0.52. No outliers are flagged. This average difference in coordinates to the GDA94 adjustment is nearly identical to the GDA2020 fixed adjustment at [ΔE: -0.091 m; ΔN: +0.082 m] and, as in the previous case study, positional uncertainty with respect to the datum can be rigorously propagated. In this survey, on average, 2D uncertainty is 0.009 m and 3D uncertainty is 0.019 m.

Traditionally, when dealing with a network of this nature, the user is forced to either float control or down-weight good measurements to fit poor control. This issue has been resolved on two fronts. Firstly, the national adjustment of all geodetic measurements provides a level of network precision never before seen in NSW, and secondly, even if this new level of precision is still insufficient for some applications, the station positional uncertainties provide a rigorous measure of absolute uncertainty with respect to datum.

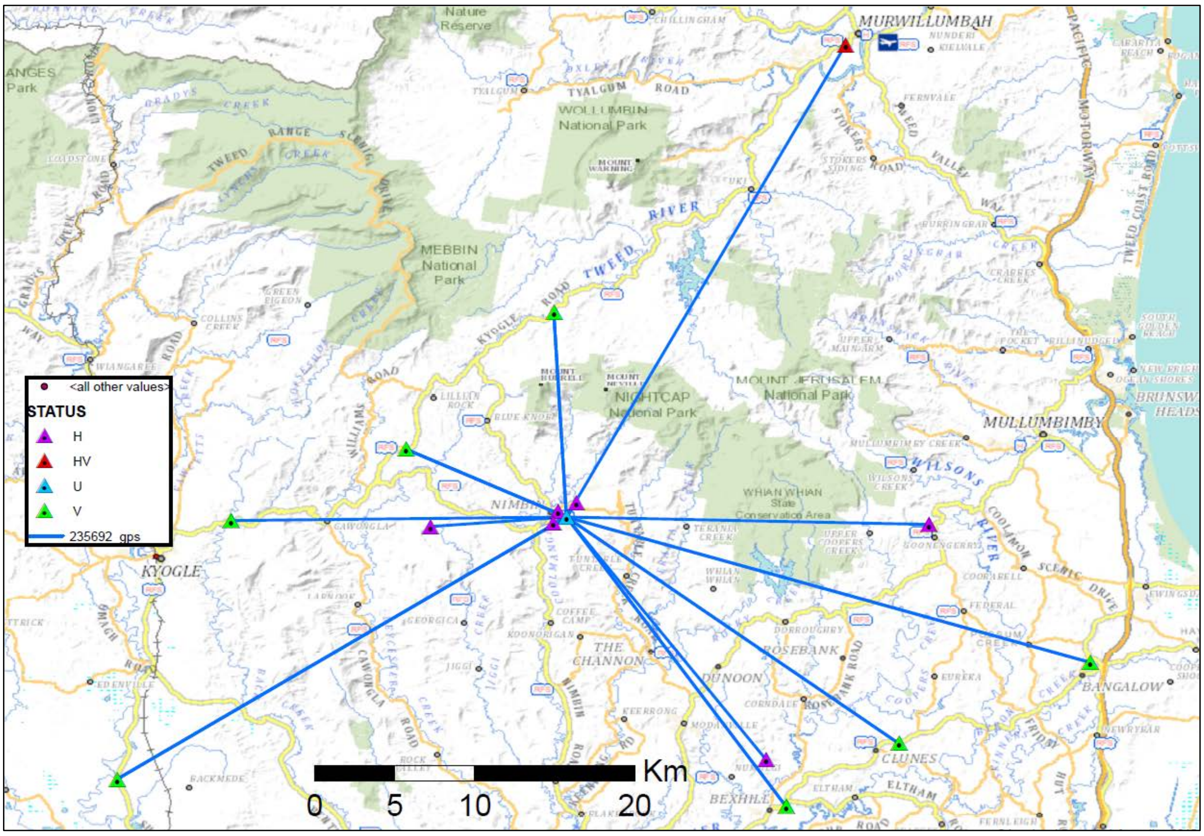

Figure 5: Nimbin CORS tie survey GNSS adjustment.

4 DISCUSSION

The GDA2020 adjustment provides a much needed rejuvenation to the NSW survey control network. The original GDA94 network was based on the coordinates of the Australian Fiducial Network (AFN) and Australian National Network (ANN) sites – 8 and 78 stations, respectively. Such a sparse control network compounded by a lack of GNSS measurements in key areas caused distortions in the order of 10 ppm or more. GDA2020 has been built upon approximately 450 CORS, several orders of magnitude better than was done for GDA94 originally.

GDA2020 is a modern CORS-based datum, densified by modern datum-focussed GNSS campaigns, GNSS CORS tie surveys, and 20 years' worth of GNSS Survey Operations networks. In effect, this rich, high-density control network provides an improved level of datum homogeneity right across NSW, as well as a seamless transition into bordering jurisdictions, which delivers an improvement of up to two orders of magnitude over GDA94.

Importantly, it is not only the numbers that have been improved. Key components of the datum modernisation campaigns have been the preservation and upgrade of survey infrastructure, including physical maintenance of permanent survey marks (including TS, PM and SS) and the update of metadata such as photographs and records (Gowans et al., 2015).

The 2020.0 epoch (i.e. 1 January 2020) aligns the National Geospatial Reference Frame to the International Terrestrial Reference Frame (ITRF) to within a few decimetres initially, and will

improve until 2020, when Australia is expected to adopt an earth-fixed reference frame as part of Stage 2 of datum modernisation (ICSM, 2017a).

The provision of rigorous uncertainty across the entire GDA2020 adjustment will permit NSW to move towards the current SP1 v2.1 for evaluation of survey control networks, although this change will not likely be available through SCIMS until a successor to the current version is available (Kinlyside, 2013).

DFSI Spatial Services is currently evaluating the provisional AUSGeoid2020 and distortion grid products. Having supplied such rich datasets, there is every expectation that the new products will provide substantial improvements over their predecessors. The results of these evaluations will be published at a later date.

5 CONCLUDING REMARKS

GDA2020 provides a much needed refresher and rejuvenation to NSW's survey control network. A single 'living' 3D network adjustment now incorporates all available geodetic-grade measurements. GDA2020 delivers significant, real, measureable improvements to NSW's survey control network and thus to its users:

- Rigorous uncertainty can now be provided across the entire survey network, and to better than 2 cm horizontal uncertainty at most stations.
- An improvement in datum homogeneity of up to two orders of magnitude is delivered while eliminating previous cross-border issues.
- GNSS CORS and passive ground marks are now aligned to with GNSS measurement precision.

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Survey Control and Quality Assurance for Aerial Imagery and Elevation Models across NSW

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ABSTRACT

Spatial Services, a unit of the Department of Finance, Services & Innovation (DFSI), provides various imagery and elevation products as part of its ongoing custodial responsibilities to the NSW Foundation Spatial Data Framework. Survey accurate control underpins each of these imagery and elevation products. This paper briefly describes the various imagery and elevation products created by Spatial Services and the accuracy requirements for these products, before examining the survey requirements, processes and practices involved in providing survey control for these products with an emphasis on working in remote locations. Finally, this paper provides a case study on the survey control capture for a large DFSI project that will provide a digital surface model for Western NSW.

KEYWORDS: *Survey control, Digital Image Acquisition System (DIAS), Light Detection And Ranging (LiDAR), GNSS, digital surface model.*

1 INTRODUCTION

The purpose of this paper is to outline the requirements, processes and guidelines related to providing survey control and quality assurance for high-resolution imagery and elevation products captured and produced state-wide by Spatial Services, a unit of the NSW Department of Finance, Services & Innovation (DFSI). Accurate and reliable orthorectified aerial imagery and high-resolution elevation data is critical to effective planning, decision making, change monitoring and risks mitigation across NSW and is utilised by government, industry and the community. Reliable survey control is fundamental to ensuring the integrity of this data which forms part of the NSW Foundation Spatial Data Framework, contributing significantly to economic, social and environmental sustainability in NSW (LPI, 2015).

The state of New South Wales is approximately 800,000 km^2 in size and consists of greatly varied terrain and bioregional landscapes including lush rainforests, rugged mountains, sandy deserts and riverine plains (NSW Office of Environment & Heritage, 2017). Providing survey control and quality assurance for imagery and elevation products across NSW presents an abundance of unique and specific challenges in terms of access, availability of marks on public record in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013), survey techniques, materials, weather and communications. A case study will be presented, outlining a specific survey that provided control and test points for imagery and elevation products, covering an area of approximately 38,000 km^2 in the Western Division of NSW.

2 BACKGROUND

The Survey Operations team functions within DFSI Spatial Services. The primary role of this team is to establish and maintain the State's fundamental survey control network via SCIMS, provide survey support to all internal aerial imagery, elevation and cadastral program and project work, and to liaise with and support other government agencies, the spatial industry and the general public.

A primary objective of the Survey Operations team is to capture, process and deliver survey control and test points to enable the processing and production of imagery and elevation products by DFSI Spatial Services. These products underpin the foundation of Spatial Data Infrastructure (SDI) in NSW and are integral for the generation of topographic maps and verification of spatial datasets within DFSI Spatial Services.

The Imagery and Elevation program and project work consists of:

- Digital Image Acquisition System (DIAS) program, which captures high-resolution 50 cm Ground Sample Distance (GSD) aerial imagery state-wide.
- Digital Town Imagery Capture (DTIC) program, which captures high-resolution 10 cm GSD aerial imagery over cities, towns and villages throughout NSW.
- Surface Model Enhancement (SME) Project, which utilises a variety of technology including aerial imagery and Light Detection And Ranging (LiDAR) to create a high-resolution, state-wide Digital Surface Model (DSM).
- LiDAR program, which captures highly accurate elevation data in high-risk areas across NSW.

3 SURVEY REQUIREMENTS FOR IMAGERY AND ELEVATION PRODUCTS

3.1 50 cm GSD State-Wide Aerial Imagery

3.1.1 Overview

DFSI Spatial Services has been undertaking aerial imagery capture cyclically across NSW since 1947, providing an invaluable dataset for the state of NSW. In order to capture NSW in a methodical and measured way, the State has been divided into 344 map sheets which are produced at a scale of 1:100,000 on A0 paper, whereby each map sheet covers 0.5º longitude by 0.5º latitude or about 54 km x 54 km (DFSI Spatial Services, 2017a), as illustrated in Figure 1. Aerial imagery is captured over 1:100,000 map sheets at a Ground Sample Distance (GSD) of 50 cm using a Leica ADS80 Airborne Digital Sensor mounted in a small aircraft.

As part of the current DIAS program, the Eastern and Central divisions of NSW are captured cyclically approximately every 7 years. As part of the SME Project, the Western Division of NSW is currently being captured with 50 cm GSD aerial imagery for the first time.

Accurate and reliable Ground Control Points (GCPs) are crucial to the orthorectification of the imagery and the accuracy of the resultant products. Orthorectified aerial imagery represents a true flat-earth image whereby each pixel represents the true X/Y/Z value on the ground at that point (Campbell and Wynne, 2011).

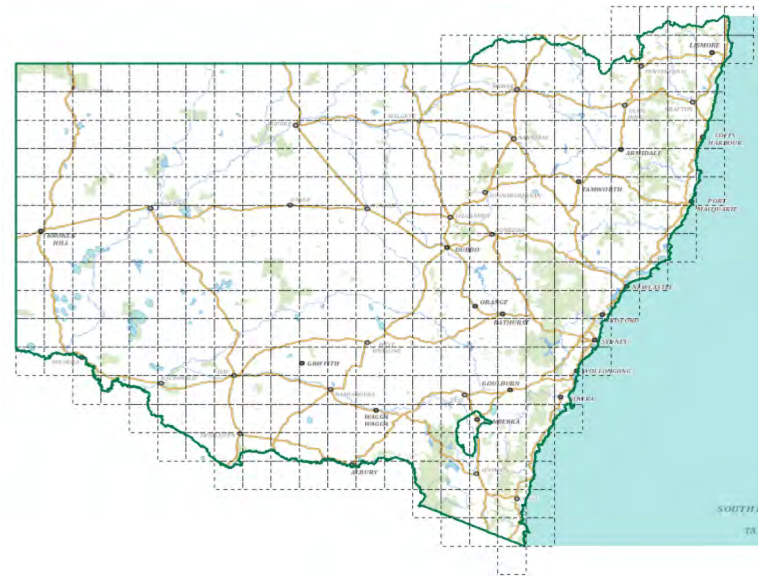

Figure 1: 1:100,000 map grid covering NSW.

Within DFSI Spatial Services, 50 cm orthorectified aerial imagery is primarily used to verify spatial data, for the creation of topographic maps and for the production of Digital Surface Models (DSMs). Externally, 50 cm orthorectified aerial imagery provides a valuable source of information for studying, monitoring, forecasting and managing natural resources, human activities and emergency events. The ongoing cyclical capture of aerial imagery across the State also provides a useful tool for viewing and comparing historical aerial imagery and verifying land uses over time.

3.1.2 Survey Control Requirements

GCPs are placed at the intersection of 1:100,000 map sheets (corners) and provide the ground control for the orthorectification of the aerial imagery. Two points are required for each corner: one primary mark which is a permanent survey mark as per the Surveyor General's Directions No. 1: Approved Permanent Marks (DFSI Spatial Services, 2016) (excluding pillars), and one secondary mark which is used for redundancy and can be a Galvanised Iron Pipe (GIP), iron spike or drill hole and is labelled as a 'CP' in SCIMS. Ideally, control points are placed within 3 km north/south of the corner and 1 km east/west of the corner to allow for the points to easily service all four map sheets.

GCPs are surveyed to an accuracy of 0.17 m horizontal (X/Y) and 0.25 m vertical (Z) with Geocentric Datum of Australia 1994 (GDA94 – see ICSM, 2014) coordinates and measured ellipsoidal height. The survey accuracy requirements are calculated based on providing control that is 3 times better than the stated accuracy of the derived products (0.5 m horizontal and 0.9 m vertical).

Processing of aerial imagery is initially performed using the panchromatic band, so a high-contrast, easily distinguishable target is critical, as illustrated in Figure 2. A 2.4 m x 2.4 m white target on a black background, placed in a cross formation and oriented true north, is placed over both survey marks. The target is oriented true north to correspond with the flight path of the plane and the resultant line of the pixels in the image which results in the target being clearly defined in the image. Targets which are not oriented true north will appear blurred in the aerial images. The size and shape of the target has been specifically chosen to be easily identifiable in the image and to allow the centre point to be accurately selected. The

materials used for the targets have been selected based on a number of factors including cost, weight, availability, ease of use and durability. Currently, targets are constructed from 3 mm whitecoat Masonite nailed over black weed mat – these materials are constantly being reviewed and alternatives discussed and evaluated.

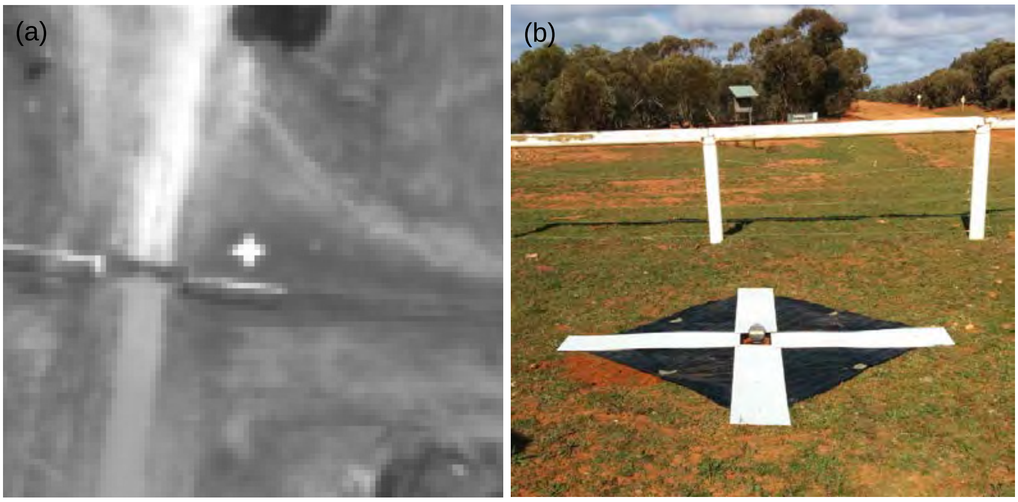

Figure 2: Ground control point (a) as viewed in 50 cm GSD aerial imagery processing software and (b) from the ground.

3.1.3 Test Points

A minimum of three Test Points (TPs) is required throughout each map sheet to check for gross errors in the orthorectification of the imagery. These TPs require an accuracy of 0.25 m horizontal and need to be identifiable in the imagery. Where possible, photo points are used for test points, i.e. existing points on the ground which are easily identifiable in an aerial image, such as the corner of a concrete slab, the corner of a large road marking (e.g. a 40 km/h sign) or the intersection of paths. Where a photo point cannot be found, a 3-wing 'T' shaped target can be placed over a survey mark, as illustrated in Figure 3. A 'T' shaped target has been selected to avoid confusion with GCPs while still being easily identified in imagery.

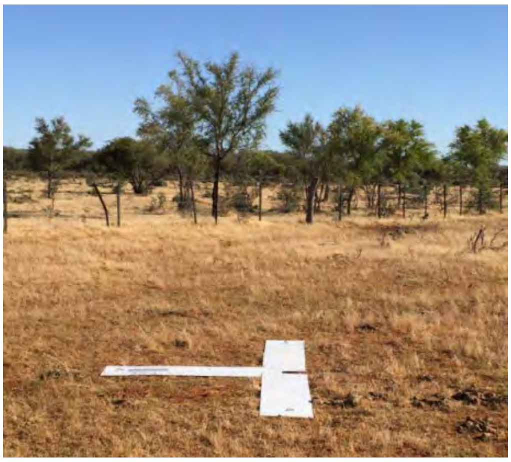

Figure 3: Example of a test point for 50 cm GSD imagery.

As is discussed in section 3.3.2, a number of test points are required to test the elevation models derived from the aerial imagery. These vertical test points can be paired with the imagery test points so that one point is able to serve two separate purposes.

3.2 10 cm GSD Town Aerial Imagery

3.2.1 Overview

As part of the Digital Town Imagery Capture Program, DFSI Spatial Services captures high-resolution digital aerial imagery for towns across NSW with a population of 400 people or greater and may capture smaller villages with significant growth or activity (Figure 4). Aerial imagery is captured over customised Areas Of Interest (AOI) at a Ground Sample Distance of 10 cm using a Leica ADS80 Airborne Digital Sensor mounted in a small aircraft. For high-population centres such as Sydney, Newcastle or Wollongong, DFSI Spatial Services may engage the private sector to assist with imagery capture and survey control to agreed specifications.

Figure 4: 10 cm GSD orthorectified aerial imagery.

The forward program for the capture of 10 cm GSD aerial imagery of towns and villages has been aligned with the capture of 50 cm imagery over 1:100,000 map sheets to allow survey work and imagery capture to be completed in one trip, and as such this imagery is updated approximately every 7 years. In areas of rapid growth or activity or where a specific request has been made, DFSI Spatial Services may elect to capture a town or village outside of this forward program. 10 cm GSD aerial imagery is a valuable resource for the whole of government and may not always be available to the general public. Local Government utilises this imagery for asset capture, planning and risk mitigation.

Due to the resolution of the imagery, it is integral that highly accurate and clearly identifiable GCPs are established for 10 cm GSD aerial imagery to ensure the usability of the resultant orthorectified imagery.

3.2.2 Survey Control Requirements

A minimum of 6 GCPs are required within the specified AOI for 10 cm aerial imagery, with one GCP per corner of the AOI (if possible) and an even distribution in the centre of the AOI. For larger towns, it is advisable to use more GCPs to provide redundancy and confidence in the end product.

Survey control for 10 cm aerial imagery requires a survey accuracy of 0.05 m horizontal (X/Y) and 0.07 m vertical (Z) with GDA94 coordinates and measured ellipsoidal height. As is the case with 50 cm GSD imagery, initial processing of the imagery is performed using the panchromatic band so a high-contrast, easily distinguishable target is critical (Figure 5). Targets for 10 cm GSD imagery are white 0.5 m x 0.5 m crosses oriented true north. Where possible, these GCPs are painted on roadways using a stencil and white exterior paint with a Galvanised Iron Nail (GIN) as the centre mark. Otherwise, a 0.5 m x 0.5 m target is constructed from whitecoat Masonite nailed over black weed mat or a photo point is selected.

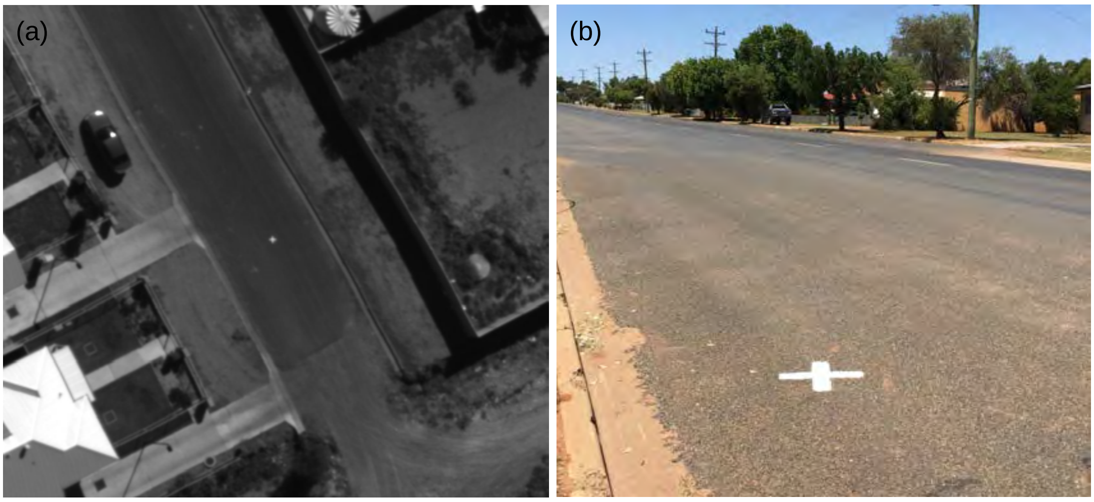

Figure 5: Ground control point (a) as viewed in 10 cm GSD aerial imagery processing software and (b) from the ground.

3.3 Surface Model Enhancement Project

3.3.1 Overview

The Surface Model Enhancement (SME) Project aims to deliver a high-resolution, state-wide Digital Surface Model (DSM) consistent with the Intergovernmental Committee on Surveying and Mapping (ICSM) Guidelines for Digital Elevation Data (ICSM, 2008). A Digital Elevation Model (DEM) will be derived from the DSM and will be used in the National Elevation Data Framework managed by ANZLIC, the Spatial Information Council (ANZLIC, 2017).

The SME Project commenced in 2014 after DFSI Spatial Services initiated a cost-benefit analysis, which identified the need for and immense benefits of a state-wide DSM. The SME Project will provide key benefits to Whole of Government including improvements in emergency services management, change detection, strategic development planning, compliance and insurance pricing, natural resource management, and law enforcement strategies.

The project integrates category 3 airborne LiDAR and surface models derived from 50 cm GSD aerial imagery to create the DSM to an accuracy of ±0.9 m Z (vertical) at 95% confidence interval and ±0.5 m horizontal at 95% confidence interval with a DEM being derived from this and output at an accuracy of 5 m. The DSM is processed and output using the 1:100,000 map sheet grid covering NSW as per 50 cm imagery (see section 3.1.1). The quality assurance of the DSM and DEM is a critical business function of the SME Project and is reliant on quality survey-accurate test points.

3.3.2 Test Points

The DSM and DEM produced are processed using ellipsoidal heights and applying a geoid model (currently AUSGeoid09) to derive the final products in the Australian Height Datum (AHD). A minimum of 6 test points are required per 1:100,000 map sheet with an even distribution across the sheet. As the geoid model is being utilised to derive an AHD product, it is critical that the accuracy of the geoid model is also tested in each map sheet. This is accomplished by establishing 'measured' AHD on a minimum of 3 test points per map sheet and comparing the values to the derived heights. These TPs require an accuracy of 0.25 m horizontal (X/Y) and 0.25 m vertical (Z). As these points are used to check the vertical accuracy of the DEM, they do not need to be identifiable in imagery and are simply an area of open, flat or uniformly sloping bare earth (Figure 6), which is located on the model using the horizontal coordinates of the point.

Figure 6: A typical test point used for testing the DEM produced for the SME Project.

Site selection is extremely important with test points used for quality assurance of elevation models as points selected near severe terrain changes or breaks in slope (e.g. culverts or structures) can result in the point being selected on the wrong side of the break line in the DEM.

3.4 LiDAR

3.4.1 Overview

Light Detection And Ranging (LiDAR) is an active remote sensing technology that uses light in the form of laser pulses to measure ranges (variable distances). Airborne LiDAR is used by DFSI Spatial Services to capture highly accurate elevation data over areas considered to be most at-risk within NSW with a particular focus on the coastline (Figure 7). It has also been utilised as part of the SME Project in areas of high vegetation or large elevation changes where the creation of a DSM from aerial imagery is impractical or where LiDAR has already been captured. LiDAR provides a superior DSM in comparison to other technologies, particularly in coastal, built up or highly vegetated areas. There are a number of limitations which prevent LiDAR technology being utilised as the sole technology for the creation of a state-wide DSM as part of the SME Project, with the major drawback being the cost of capture and the timeframe required for capturing and processing this data.

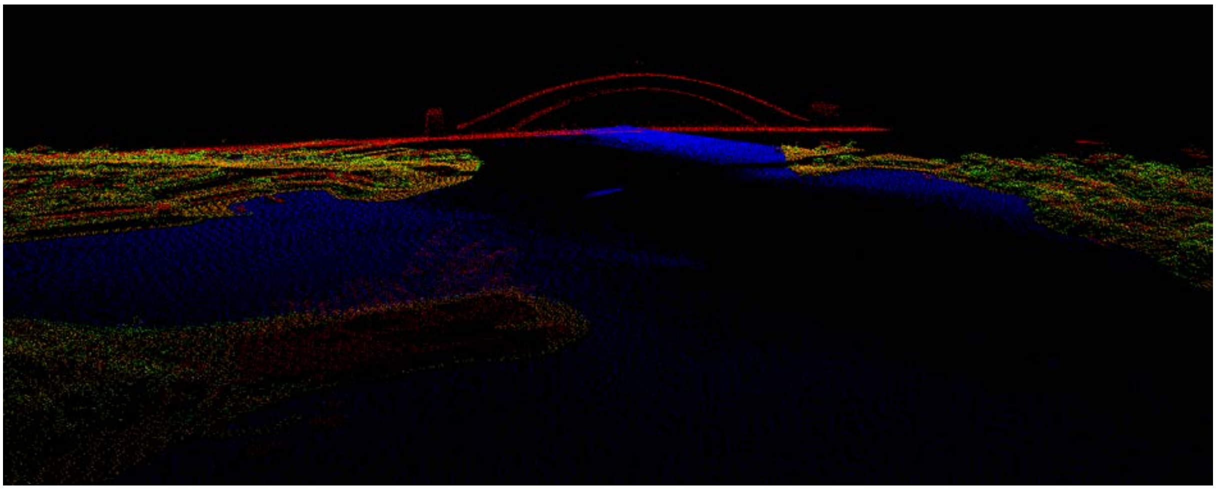

Figure 7: Classified LiDAR point cloud as captured over Sydney Harbour Bridge.

DFSI Spatial Services uses a Leica ALS50 Airborne LiDAR Sensor mounted in a small aircraft to capture point cloud data at a range of point densities and levels of classification in accordance with the ICSM Guidelines for Digital Elevation Data (ICSM, 2008). Category 1 LiDAR boasts a vertical accuracy of ±0.3 m at 95% confidence interval with one point captured per square metre and is primarily used for capturing elevation data over areas which have been identified as high risk. Category 3 LiDAR is captured at one point per four square metres which creates a DSM with vertical accuracy of ±0.9 m at 95% confidence interval and is utilised in the SME Project. Understandably, accurate and reliable test points are fundamental for determining the vertical accuracy of the LiDAR.

3.4.2 Test Points

Specifications for test points for category 1 LiDAR are given in the 'Elevation Products Data Specifications and Descriptions No. 2' written by DFSI Spatial Services in alignment with ICSM (2008). A minimum of 5 points per 1,000 km^2 with both an accurate AHD height (SCIMS Class B/LD or better) and measured ellipsoidal height. As is the case with SME Project test points, these points do not require any physical marking on the ground as they are testing a point cloud. Test points need to be in open, flat or uniformly sloping bare earth (see Figure 6), with a horizontal coordinate provided to an accuracy of 0.5 m used to locate the point in the cloud. The accuracy requirements for test points for category 3 LiDAR are the same as those required for the SME Project (see section 3.3.2).

4 CASE STUDY

4.1 Overview

As part of the SME Project, the Survey Operations team was tasked with providing survey control and test points for 1:100,000 map sheets being captured with 50 cm GSD aerial imagery in the Western Division of NSW. The area to be captured consisted of over 120 map sheets covering approximately 350,000 km^2 or 40% of NSW and included some of the most remote parts of the State.

The SME Project is a 5-year project that commenced in 2014, and the capture and processing of 50 cm GSD aerial imagery is an integral component to the success of the project. There are

a number of factors which affect the capture of imagery across the State and these had to be factored into a 4-year forward plan for providing the survey control, imagery capture and output within the set time frames. Key factors that were taken into account on this plan were:

- Solar altitude: In the Central and Western Division of NSW, aerial imagery can only be captured when the solar altitude is >35º and in the Eastern Division >40º in order to minimise shadows in the imagery. Consequently, a solar altitude map has been constructed to assist with planning (Figure 8).
- Longevity of GCPs: It is recommended that imagery is captured within 3 months of placing GCPs to guarantee the quality of the points. However, targets can last much longer in good circumstances.
- Weather: Aerial imagery can only be captured when there are no clouds in the sky so as to prevent shadows in the imagery. Due to the Western Division of NSW consisting of 'dry weather only' roads, the Survey Operations team is limited by weather with roads being closed in wet weather. For Work Health and Safety reasons, the Survey Operations team does not work in the far west of the State during summer due to extreme temperatures.
- Resources: Field work for survey control is captured over 2-week periods and is resource intensive in terms of team members, vehicles, equipment and materials, so this must be factored in during planning.

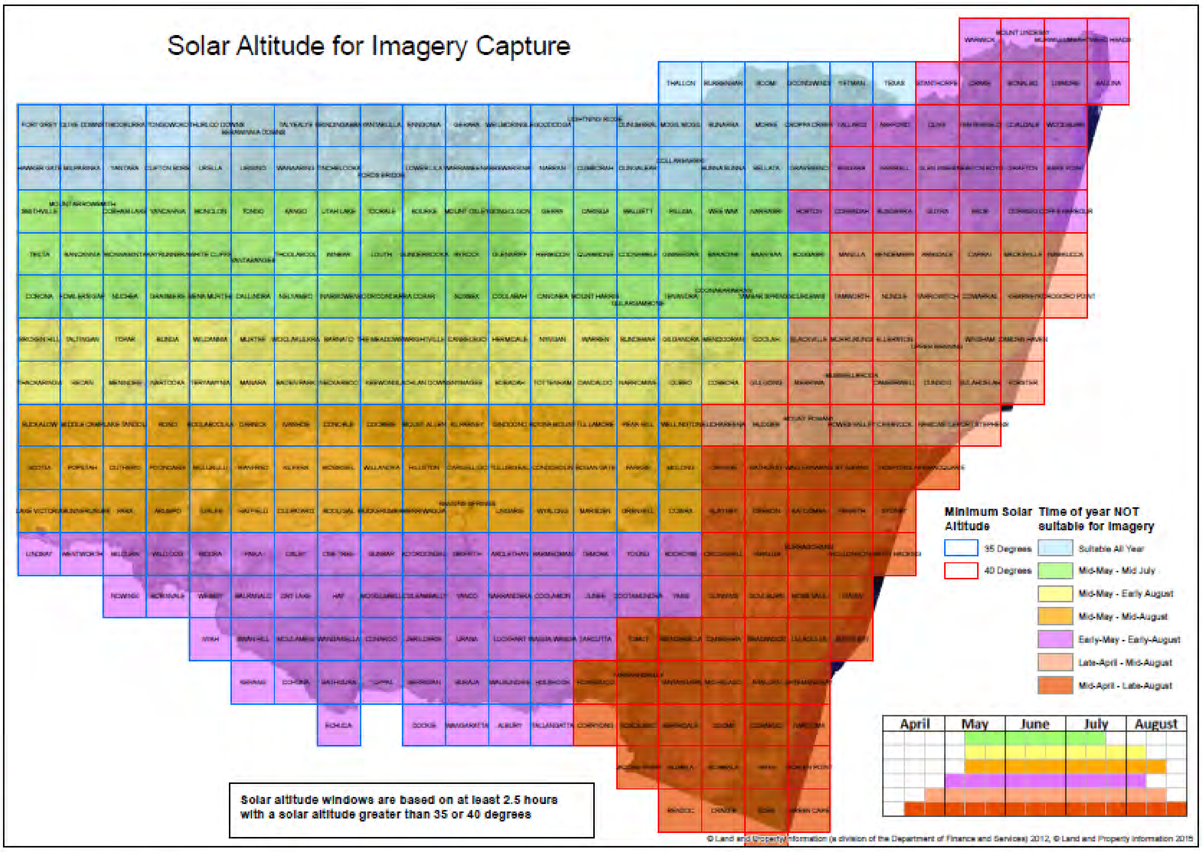

Figure 8: Solar altitude map for imagery capture over NSW.

The Survey Operations team, in co-operation with the Imagery and Elevation team and the SME Project team, created a preliminary forward plan for survey ground control and imagery capture which saw the area broken up into blocks of 10-12 map sheets to be captured in spring, winter or autumn. The forward plan (Figure 9) is flexible and has been amended as the project progressed. Where conditions such as weather, access and availability of resources

were ideal, more sheets were captured (up to 16 in one trip). High rainfall over the winter of 2016 resulted in delays for capturing survey control and imagery and as a result the plan has been amended to allow extra field work to be undertaken in 2017 within the deadlines of the project.

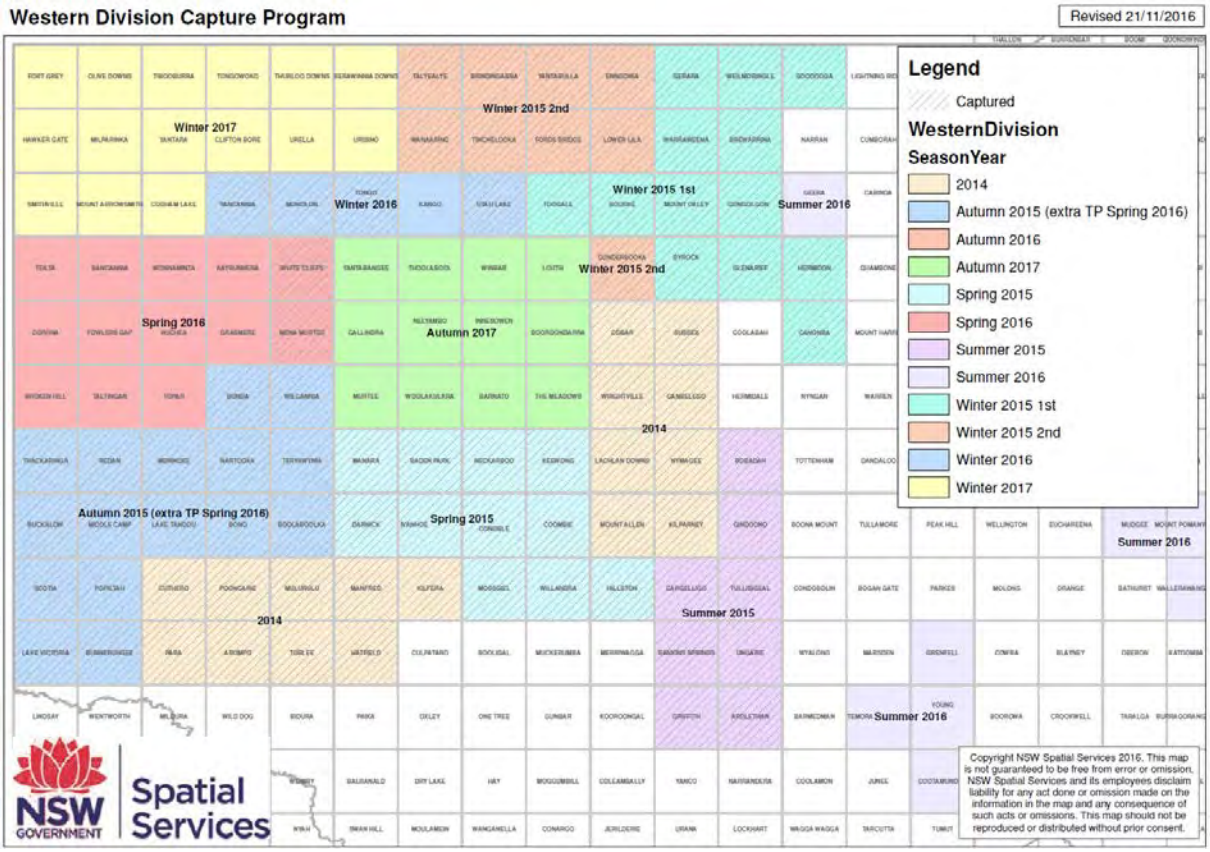

Figure 9: The Western Division Capture Program as of November 2016.

4.2 The Task

The job, labelled ‘Spring 2016’ in the ‘Western Division Forward Plan’ (see Figure 9), covered an area of approximately 38,000 km^2 and was located in the Unincorporated, Broken Hill and Central Darling Local Government Areas, including the townships of White Cliffs, Packsaddle and Broken Hill (Figure 10). The job was a day’s travel from the office in Bathurst and was completed using an 8-person survey crew with four vehicles.

The scope of the job was to provide ground control and test points for thirteen 1:100,000 map sheets to be captured with 50 cm GSD aerial imagery as well as providing test points for the derived DEM. In addition, the team was asked to provide survey control for one town to be captured with 10 cm GSD aerial imagery and to complete a Global Navigation Satellite System (GNSS) Continuously Operating Reference Station (CORS) tie survey (Gowans and Grinter, 2013) whilst in the area.

Initially, the survey work was scheduled for September 2016 to align with the Aerial Survey Unit (ASU) schedule for imagery capture. However, this was delayed until November 2016 due to ongoing rainfall, resulting in many parts of the job being inaccessible.

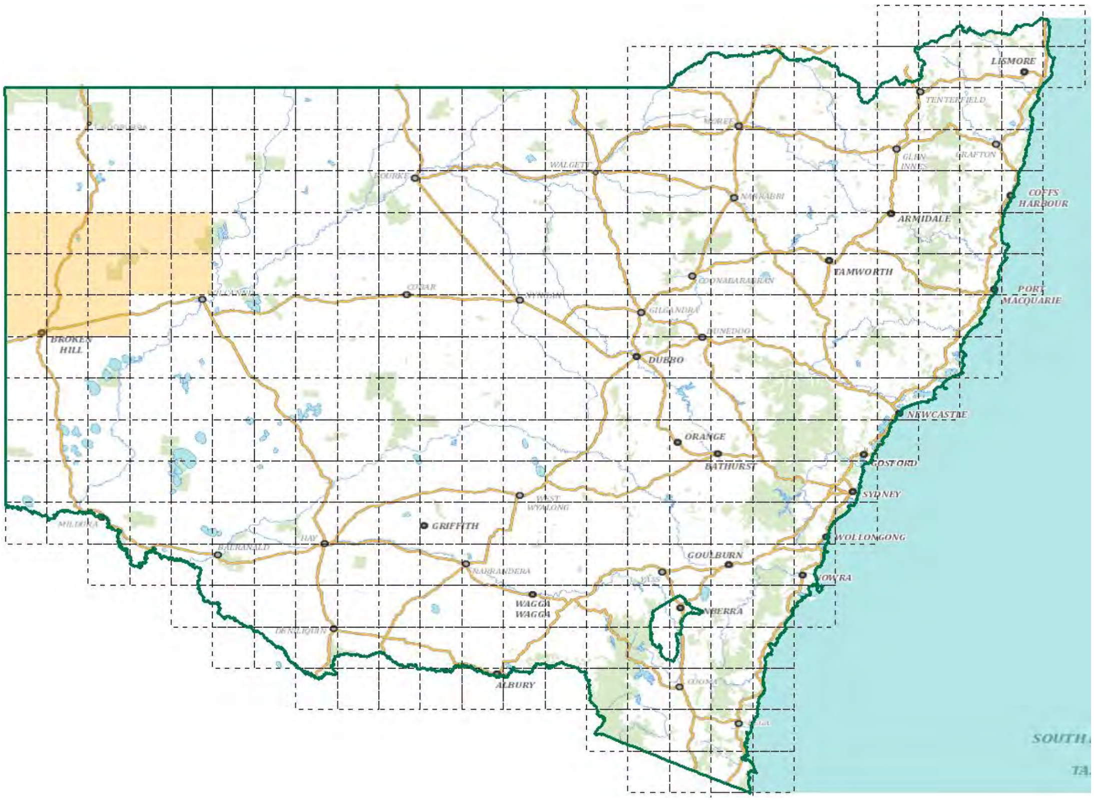

Figure 10: Location of the required survey control within NSW.

4.3 Survey Requirements

Due to the configuration of the 13 map sheets, there were a total of 22 corners requiring two ground control points each. Of this, 9 corners had not previously been surveyed, which meant that an existing survey mark would have to be located or Permanent Marks placed and surveyed for each of these corners. 18 of the corners were located on private property, so land owners had to be contacted and permission gained to access these locations. Static GNSS baselines from established marks and AUSPOS solutions (GA, 2017) were utilised for gaining horizontal position and ellipsoidal heights on new marks.

A minimum of 6 vertical test points were required for each map sheet to test the DEM to be derived from the imagery for the SME Project, which equated to 78 points evenly distributed throughout the job requiring measured ellipsoidal height. At least half of these also required measured AHD to verify the geoid model in this area. Three test points per map sheet also required physical marking on the ground that is identifiable in an aerial image to test the orthorectification of the imagery, equating to 39 marks with a ‘T’ shaped target.

The township of White Cliffs was identified as requiring 10 cm GSD aerial imagery as it had not been previously captured with high-resolution aerial imagery. Targets were placed by the aerial survey unit on a previous job, and as such only survey work was required for these points. The Real-Time Kinematic (RTK) GNSS technique was utilised for establishing these marks, using CORSnet-NSW’s White Cliffs CORS as reference station.

Fowlers Gap CORS was constructed in October 2016 as part of the continuing expansion of the CORSnet-NSW positioning infrastructure (Janssen et al., 2016; DFSI Spatial Services, 2017). Due to the remote location of this CORS, it made sense to incorporate the local tie survey (to connect the CORS to existing surrounding ground control) into this job.

4.4 Planning

Due to the variety and scale of work to be undertaken as well as the remote location of the job, thorough planning and preperation was critical to undetaking the work in an efficient and safe manner whilst delivering within set time frames. A survey party leader was appointed the task of organising the planning, capture, processing and delivery of this job.

Initial planning of the job identified minimal survey control in the area, especially in horizontal position and with two map sheets having no AHD marks within them (Figure 11). Furthermore, 8 of the 13 map sheets contained 'dry-weather only' dirt roads and 2 of these did not contain any public roads. Gaining access to all of the required locations throughout the map sheets and establishing survey control to the accuracies required for 50 cm GSD imagery and the SME Project was the first consideration in planning this job, followed by predicted travel times to and between marks, required materials and resources.

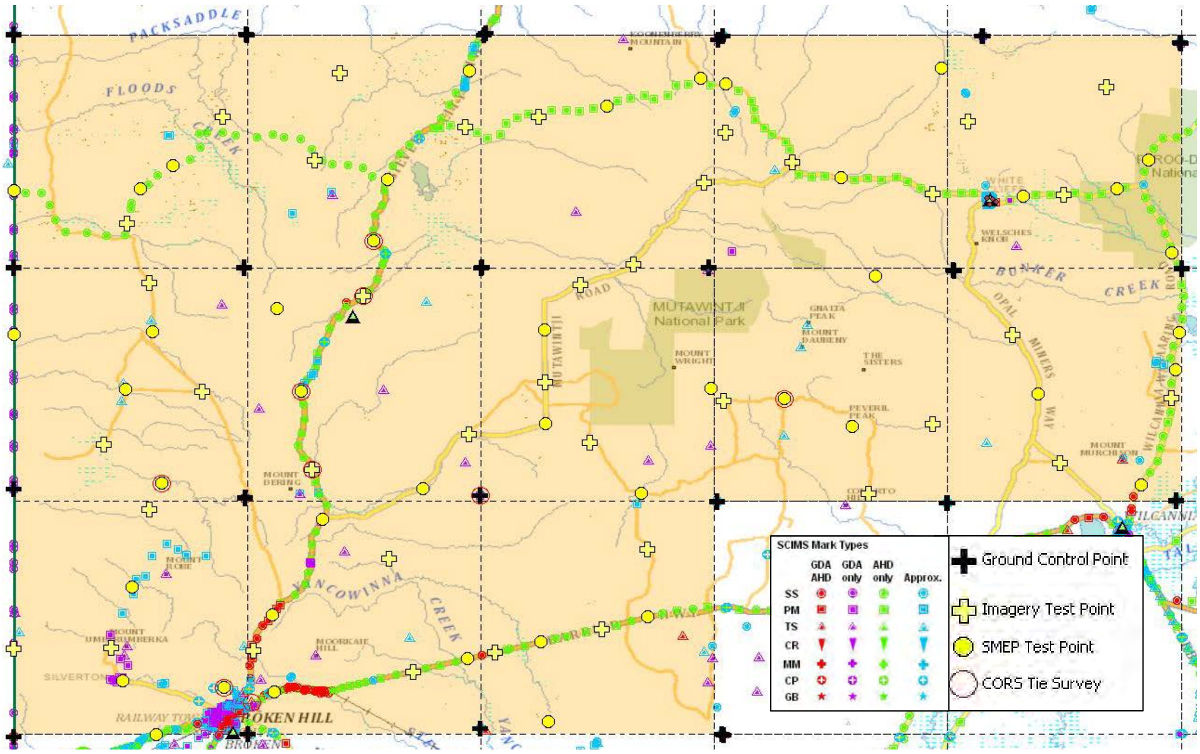

Figure 11: Planning location and survey for Ground Control Points and Test Points throughout the AOI.

Using ArcMap, a shapefile was created which identified desirable locations for GCPs and TPs, attribute data to be assigned to each point in regard to the survey required for it, whether it was on private property and, if so, contact details for the land owner. This file was used to create hard copy maps as well as being loaded on to ArcPad on tough book computers to be taken into the field and used for navigation.

The survey was designed utilising static GNSS baselines, AUSPOS solutions, RTK/NRTK and differential levelling. Each mark had different survey requirements depending on its use and location within the job.

A preliminary plan was designed to allow the survey control to be established in the safest and most efficient way. This was used to allow accommodation to be booked and land owners to be contacted and given expected dates of the work. The required paperwork included

Locality Sketch Plans, control point information, owner details and maps in order to allow for planning to be adapted during the course of the field trip as work progresses faster or slower than anticipated.

4.5 Resources

The job was undertaken in a 2-week window by an 8-person field party consisting of four Surveyors and four Survey Assistants, working in teams of two. Four vehicles were used, each equipped with long range fuel tanks, driving lights, HF radios and winches to allow work to be undertaken safely in remote locations (Figure 12).

Figure 12: Vehicles fuelling up in Packsaddle.

Each team was equipped with 5 GNSS receivers, 5 tripods, a spirit level, fixed tripod, levelling staff, bipod, hand tools, first aid kit, vehicle recovery kit, personal locator beacon, Toughbook computer and GPS navigator. Two brush cutters and two Satellite SPOT rovers were also taken and allocated to teams based on work requirements each day.

Materials were purchased prior to the trip and taken to the job in a trailer. This included 38 sheets of 3 mm Masonite cut into 300 1.2 m x 0.3 m ‘wings’, 4 rolls of weed mat, 4 boxes of 150 mm flat head nails, 120 star pickets for protecting survey marks, 80 Galvanised Iron Pipes, 12 cover boxes and 12 galvanised iron star pickets for placing Permanent Marks.

4.6 Setbacks and Challenges

Weather is a major concern when planning work in the Western Division of NSW as rain can result in road closures for several weeks and can leave properties inaccessible for much longer. The trip was initially scheduled for the start of October 2016, and 5-10 mm of rain was predicted in the first week of the trip. Fortunately, the rain arrived on the Sunday before the trip commenced and was much higher than expected, which resulted in road closures for over a week and several properties being inaccessible for up to a month. As a result, the trip was postponed until November – by then the roads had been re-graded and properties were accessible.

Placing ground control points within a few kilometres of the intersection (corners) of 1:100,000 map sheets often poses a challenge in terms of access as these corners can be located on private property, in protected areas (e.g. National Park or conservation area), in rugged terrain or high vegetation areas. Identifying land owners and gaining permission to place control points on private property is a critical step in planning for this type of work.

Within this job, there were two land owners that could not be contacted prior to the trip. In both cases, the solution was to call the neighbouring properties and ask for assistance in making contact with the land owners before going on the property.

As mobile phone reception is scarce in this part of the State, high-frequency (HF) radios are utilised for communication between the survey parties. Several days into the trip one of the radios malfunctioned and could not be repaired, resulting in one field team having minimal communications. In order to mitigate this issue, the team with minimal communications were tasked with work along major roads and in areas of known mobile phone reception, were given a SPOT satellite navigation unit (allowing them to be tracked and to send distress signals). Where possible, this team did not undertake survey work that required static GNSS baselines to other parties.

Issues with access to corners that have not been surveyed previously can often cause major delays as roads or tracks can be impassable, non-existent or extremely rough. One particular map sheet in this job had had new roads graded since the last topographic maps were made and as a result the roads that were shown on maps no longer existed, resulting in a 4-hour delay for one team as they navigated to the required points. Using HF radio communication with a nearby team, the impact of this delay was able to be minimised as work could be redistributed between teams.

Large numbers of wildlife and livestock on roads and tracks in the Western Division of NSW present a constant hazard when driving between survey marks (Figure 13). By ensuring that work is concluded before dusk each day and teams drive to the conditions and with caution, the likelihood of a collision with an animal is minimised. However, it is impossible to eliminate this risk. Unfortunately, during the course of this job, an emu hit a stationary vehicle causing cosmetic damages to the vehicle.

Figure 13: Animals present a constant hazard on roads in the Western Division of NSW.

Upon processing and performing a least squares network adjustment using the data collected on this job, it was found that there was a number of disagreements between established GDA94 horizontal marks and also between established AHD marks throughout the network. Further analysis and amended network design was utilised to identify survey marks which had suffered movement and no longer agreed with surrounding marks.

4.7 Deliverables

Due to favourable weather conditions, straight forward accesses, the ability to utilise survey work completed by DFSI Spatial Services staff earlier in the year and minimal equipment

malfunctions or other setbacks, the job was finished 4 days ahead of schedule. As a result, the field party was able to complete extra SME Project test points for 12 map sheets south of this job as well as a local tie survey for Ivanhoe 2 CORS. In total, 13 map sheets were provided with survey control for 50 cm GSD aerial imagery, 25 map sheets were provided with test points for quality assurance of the DEM for the SME Project, one town was provided with survey control for 10 cm GSD aerial imagery, and two local tie surveys were undertaken for newly built CORSnet-NSW stations.

Post-processing of survey data was carried out in the office using Leica Geomatics Office, and two least squares network adjustments were performed to establish and verify horizontal position, AHD height and ellipsoidal height for all GCPs and TPs. All 1+ hour GNSS sessions were submitted to AUSPOS (GA, 2017), and all 6+ hour GNSS sessions were submitted to be utilised in the national GDA2020 adjustment in order to contribute to Australian datum modernisation efforts (Gowans, 2017). A survey report was written, SCIMS was updated and the required results were delivered to the Imagery and Elevation team and the SME Project via a 'Control Point' database in ArcMap.

5 CONCLUDING REMARKS

On behalf of the Surveyor General, DFSI Spatial Services has a legislative, regulative responsibility to maintain the geodetic control network in NSW. Accurate and reliable survey control is essential to the production of high-resolution orthorectified imagery and high-accuracy elevation models produced by DFSI Spatial Services as part of the NSW Foundation Spatial Data Framework. Imagery and elevation products produced by DFSI Spatial Services are highly beneficial resources for both government and private stakeholders and are necessary for supporting analytical and planning functions across NSW.

The Survey Operations team, in conjunction with the Imagery and Elevation team, has developed standards and guidelines for delivering ground control points and test points in accordance with the ICSM guidelines to ensure the accuracy specification of imagery and elevation products is surpassed.

This paper has outlined that providing ground control points and test points across NSW for imagery and elevation programs and projects presents an abundance of unique and specific challenges, which can be addressed and mitigated through thorough planning and preparation. Capturing the Western Division of NSW with aerial imagery as part of the Surface Model Enhancement Project has allowed the Survey Operations team to develop a number of strategies for working in remote locations where communication, vehicular access and survey control are all limited and weather can cause significant setbacks.

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Investigating an Anomaly in the Australian Height Datum at the NSW-Victoria Border in Barham/Koondrook

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ABSTRACT

The Australian Height Datum (AHD) is the national vertical datum for Australia. AHD continues to be a practical height datum that supports many surveying and engineering applications. However, several anomalies exist in the national height datum. One such anomaly was identified at the border between New South Wales and Victoria at Barham (NSW) and Koondrook (VIC). This discrepancy, of approximately 0.14 m, has major implications for surveyors and spatial professionals working on either side of the state border, as well as for flood management of the Murray River. This paper outlines the collaborative work performed by DFSI Spatial Services and the Office of the Surveyor-General Victoria to investigate and resolve this anomaly, using Global Navigation Satellite System (GNSS) technology and conventional 2-way levelling that included crossing several rivers. It was found that recent levelling conducted on both sides of the border and allowance for suspected mark instability at a national junction point in Victoria reduced the discrepancy to 0.066 m. The remaining anomaly can be attributed to the accumulation and distribution of error in the original data used to propagate AHD across Australia. Combining the NSW and Victorian level data into one contiguous adjustment, linking the junction points in Moulamein (NSW) and Kerang (VIC), will allow the remaining discrepancy to be distributed across the entire level run, ensuring much closer harmony between AHD marks on either side of the Murray River in Barham/Koondrook.

KEYWORDS: *AHD, levelling, GNSS, height datum anomaly, interstate collaboration.*

1 INTRODUCTION

Spatial Services, a unit of the NSW Department of Finance, Services & Innovation (DFSI), has a legislative, regulative responsibility to maintain the geodetic control network across the State on behalf of the Surveyor General of New South Wales. As such, DFSI Spatial Services

is the custodian of more than 250,000 marks in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013), which includes about 100,000 level marks.

The Australian Height Datum (AHD) is the national vertical datum for Australia. It was defined by assigning zero to the average Mean Sea Level (MSL) values recorded from 1966-68 at 32 tide gauges located around Australia. This served as constraint in an adjustment of 97,230 km of 2-way spirit levelling across Australia (Roelse et al., 1975). Now, 50 years later, it is well known that shortcomings in the AHD realisation (AHD71 for mainland Australia and AHD83 for Tasmania) resulted in MSL not being coincident with the geoid at the tide gauges involved.

These shortcomings included not considering dynamic ocean effects (e.g. winds, currents, atmospheric pressure, temperature and salinity), a lack of long-term tide gauge data, and the omission of observed gravity. This has introduced considerable distortions of up to about 1.5 m into AHD across Australia (e.g. Featherstone and Filmer, 2012). However, AHD continues to be a practical height datum that provides a sufficient approximation of the geoid for many surveying and engineering applications. Consequently, in surveying and engineering practice, AHD heights are often accepted as being equivalent to orthometric heights.

Nevertheless, several anomalies exist in the national height datum. One such anomaly was identified at the border between New South Wales and Victoria at Barham (NSW) and Koondrook (VIC). This discrepancy, which is approximately 0.14 m in magnitude with NSW heights being lower than Victorian heights, has major implications for surveyors and spatial professionals working on either side of the border, as well as for flood management of the Murray River. This paper outlines the collaborative work performed by DFSI Spatial Services and the Office of the Surveyor-General Victoria to investigate and resolve this anomaly, using Global Navigation Satellite System (GNSS) technology and conventional 2-way levelling that included crossing several rivers.

2 BACKGROUND

In 1979, Water Resources conducted levelling that stretched from Deniliquin to Kyalite, near Balranald, with an offshoot to Barham from a junction point near Wakool (Figure 1). Following an adjustment of this levelling in 1980, it was noticed that there was a 'half-a-foot' misclose between NSW and Victorian levelling at Barham.

As data for the Water Resources connection from Barham to the national levelling route at Kerang (VIC) was unavailable, the adjustment adopted the State Rivers & Water Supply Commission's (SR&WSC) heights at Barham as "AHD heights of unknown origin from SR&WSC (VIC)". At the time, it was decided to adjust out the misclose over a larger area, by spreading it as far back as Kyalite and Deniliquin. Due to constraints within the adjustment software (LEVADJ) at the time, the level run was split into two runs: one from Barham to Kyalite (TBM1138 – SS4264), and a branch to Deniliquin (TBM1038 – SS4809 & SS4815). This resulted in a 0.13 m misclose between Barham and Kyalite and a 0.12 m misclose in the branch to Deniliquin.

In 1994, the adjustment was revisited following an amendment to the LEVADJ software. Although the amendment made it possible to adjust the entire adjustment as one, it was decided to adjust the levelling between Deniliquin and Kyalite, which resulted in a mere 15

mm misclose over about 185 km, and a spur of 28 km from TBM1038 to Barham (see Figure 1).

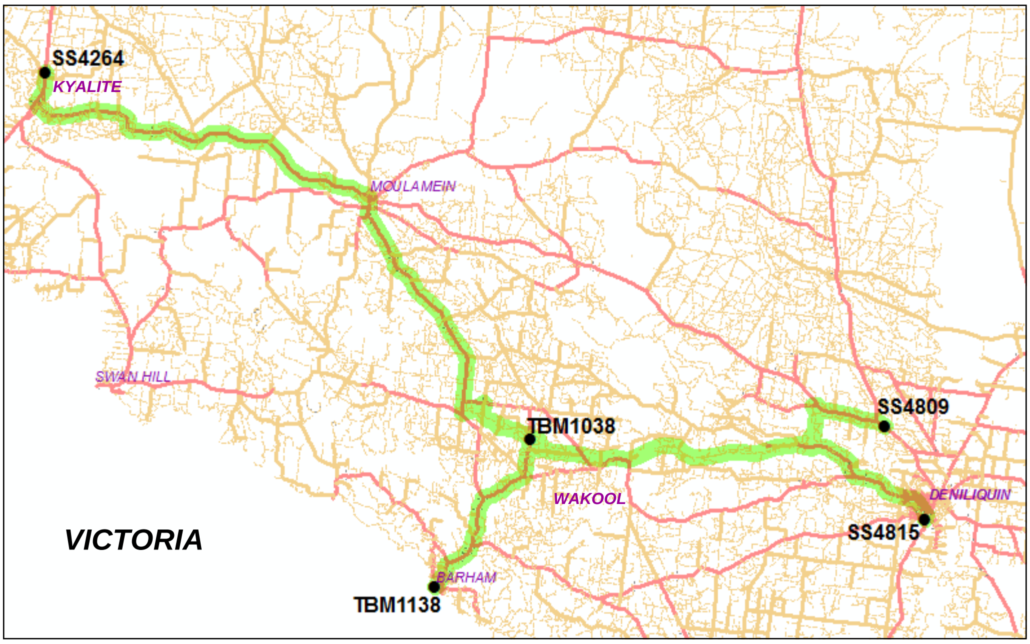

Figure 1: Water Resources levelling performed in 1979, highlighted in green.

It was also decided to not constrain the SR&WSC marks in Barham, as the connection to Victorian control was regarded as non-geodetic. This meant that the spur to Barham was unclosed and, as a result, the heights in Barham dropped by half a foot.

This shift in the datum was noticed in the late 1990s by Russell Douthat of Laughlin Surveys, Barham, who was involved in supplying site levels for new dwellings in the Barham/Koondrook area. These site levels were then compared to existing flood level information, which he believed to be based on the pre-1994 height datum. Mr Douthat was concerned about their (and other surveyors') legal liability in regards to problems arising from the changed datum as well as the inhomogeneity of the height datum between NSW and Victoria.

After much correspondence with Mr Douthat in the late 1990s to early 2000s, it was attempted to source information on the levelling of the SR&WSC marks in Barham from Goulburn Murray Water. It was hoped that access to this levelling information would enable another readjustment of the levelling, constraining the height in Barham to Victorian derived AHD. Unfortunately, these attempts proved to be unsuccessful.

The Department of Sustainability and Environment (VIC), which is now the Department of Environment, Land, Water & Planning, was subsequently contacted to attempt to resolve the discrepancy at Barham. While reduced levels (RLs) of a levelling connection from the national levelling route at Kerang to Barham were provided, the origin of the levelling information remained unknown.

Although the existence of this discrepancy had been known for many years, nothing substantial was done to rectify the issue, primarily due to a lack of resources. However, in 2012/13, an opportunity to conduct some investigative levelling arose with the dual benefit of transferring the necessary knowledge base to younger survey staff at DFSI Spatial Services.

3 FIELD WORK AND OBSERVATIONS IN NSW

3.1 Initial Reconnaissance

Initial reconnaissance was carried out on 20-24 February 2012, with the purpose of confirming the presence of the discrepancy and attempting to isolate the location of a potential error. Firstly, in order to confirm the discrepancy between NSW and Victoria, spirit levelling was conducted on either side of the state border. A simultaneous and reciprocal trigonometrical heighting methodology was adopted to transfer height across the Murray River (between PM22423 and PM148306). Figure 2 illustrates this initial stage of the reconnaissance survey.

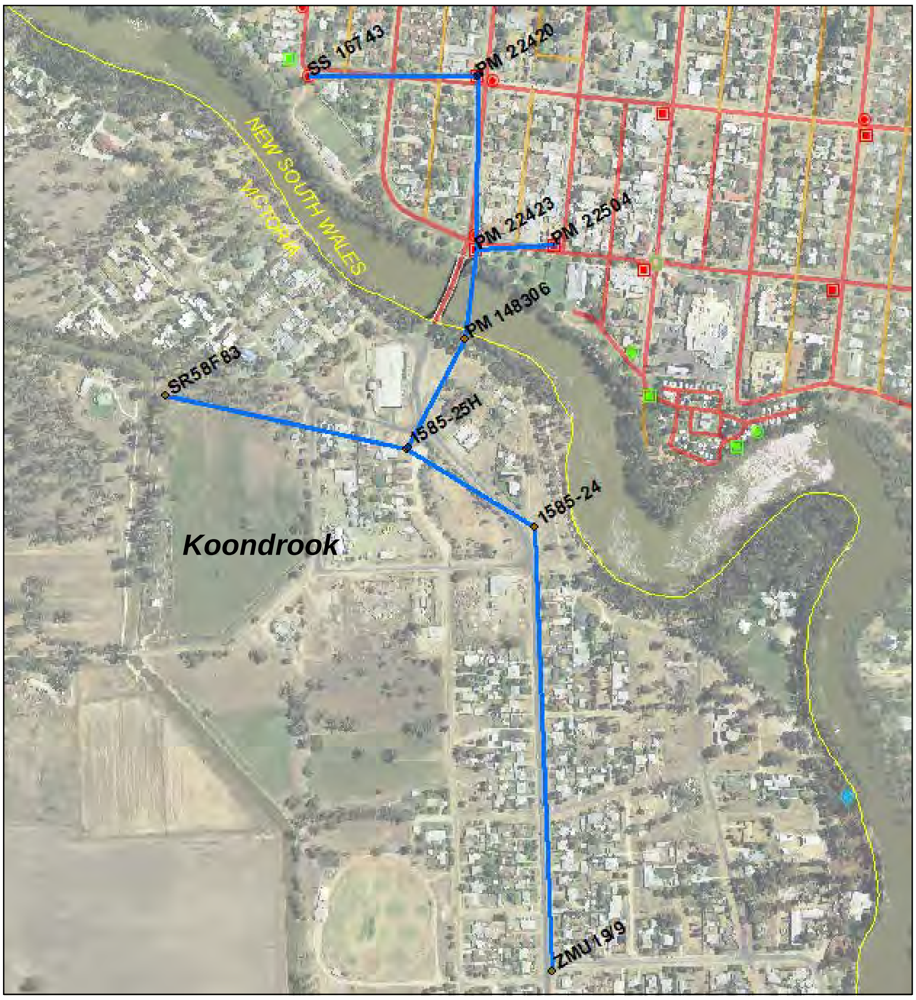

Figure 2: Reconnaissance levelling performed in February 2012, indicated in blue.

The NSW-derived heights for the four marks in Koondrook were found to be about 0.11-0.15 m lower than the Victorian heights. Although not particularly rigorous, this survey indeed confirmed the discrepancy in the AHD between NSW and Victoria.

Secondly, an extended GNSS survey was conducted in an attempt to identify the source of the error so that further investigative survey work could be better focused. Static GNSS baselines were measured to connect the levelling in Barham/Koondrook to the national levelling route at Kerang as well as to two marks (TBM1125 and TBM1119) along the original Water Resources levelling route. Figure 3 illustrates the observed GNSS network. Baseline lengths

ranged between 600 m and 15 km, and observation sessions lasted between 8 and 49 minutes, generally depending on baseline length. All 10 marks were also occupied using Real Time Kinematic (RTK).

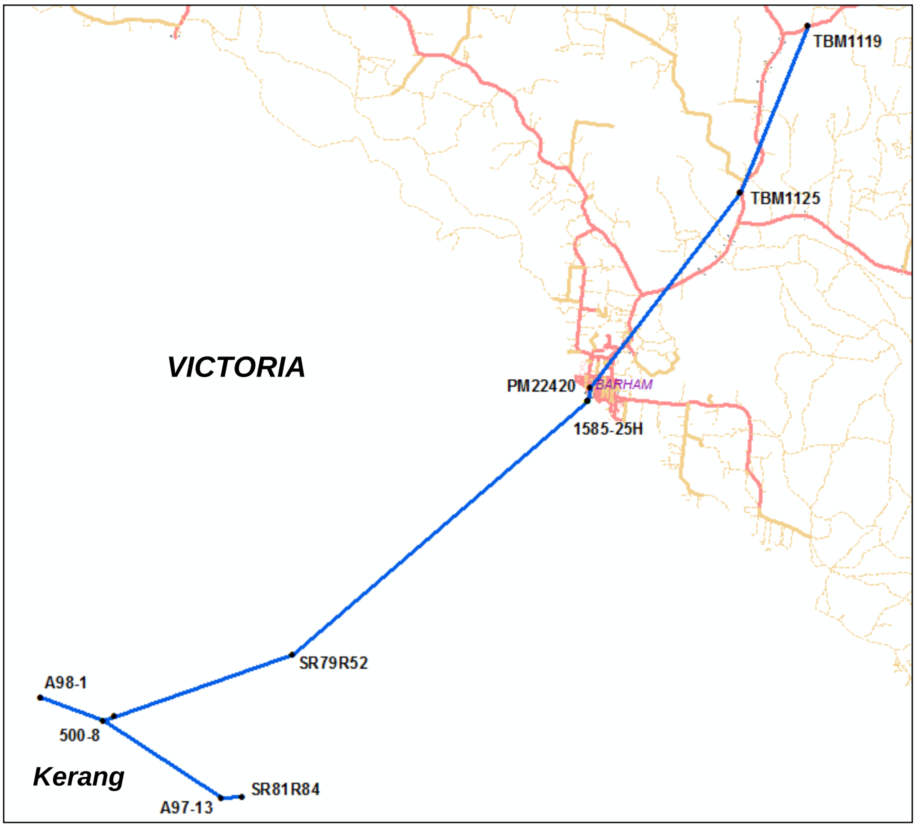

Figure 3: Static GNSS measurements (February 2012).

By constraining the height of PM22420, in Barham, it was clearly evident that the discrepancy noticed at the border extended to the national levelling marks in Kerang with differences between NSW and Victorian heights again ranging from 0.09 m to 0.13 m. At the same time, the derived heights of the Water Resources marks agreed with original levelling data to within expected values based on the methodology applied and the soil composition in the area.

This led to the conclusion that the error could lie within the original Water Resources levelling, most likely in the 28 km spur from TBM1038 to Barham. Attempts were made to locate TBM1038 (nail in post) and other benchmarks near this junction but were unsuccessful.

3.2 Secondary Reconnaissance

After the initial reconnaissance uncovered that the height datum discrepancy at the NSW-Victoria border was likely caused by an error within the original levelling from Wakool to Barham, it was decided that re-levelling the offshoot to Barham was necessary to resolve the discrepancy in the AHD.

However, as the benchmarks at the junction of the spur to Barham (TBM1038) were not located in the initial reconnaissance, it was decided that in order to re-establish a reliable datum it would be necessary to level to Barham from as far back as Moulamein. Office investigations also noted a third-order levelling connection from TS3479 (also known as

national junction point 1873), 8 km north of Moulamein, to the national levelling route between Hay and Balranald.

A secondary reconnaissance was conducted on 12-17 August 2013 with the purpose of locating the original Water Resources benchmarks and NSW state control marks (PMs and SSMs) between Moulamein and Barham. Each mark was then coordinated horizontally at the sub-metre level using Differential GPS (DGPS) technology in order to facilitate easy future access.

During this reconnaissance, it was also decided to level between TS3479 and SS16734 (in Moulamein) to confirm the original Water Resources levelling datum at Moulamein. Due to time constraints the levelling was only performed one-way using a Leica DNA03 and fiberglass telescopic staff. The resulting height difference between TS3479 and SS16734 showed 1 mm agreement with published values.

3.3 Levelling Survey

The levelling methodology adopted follows the recommendations given in ICSM (2007, 2014). Second-order levelling was conducted in two separate stages: the first on 16-27 September 2013 and the second on 4-9 November 2013. Levelling was performed by two field parties of three members, each using a Leica DNA03 digital level on a fixed-leg tripod sighting to two calibrated 3-metre rigid invar staves. The two field parties levelled in opposite directions to one another.

The verticality of the spot bubbles on the invar staves was checked prior to each of the levelling trips and no adjustments were needed. The collimation of the digital level was checked daily prior to the commencement of each day's levelling via a two-peg test using the 'A x x B' method, i.e. using two instrument setups between the two staff positions (Figure 4). The results of the collimation check were stored in the instrument each day.

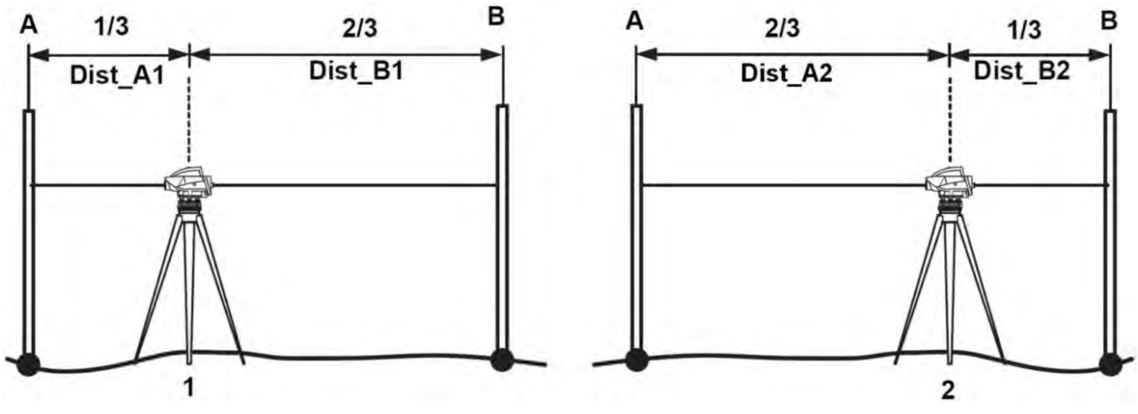

Figure 4: Two-peg test setup following the 'A x x B' method (Leica Geosystems, 2006).

All levelling observations were made using the 'B-F-F-B' methodology (i.e. backsight-foresight-foresight-backsight). Each recorded observation was the mean of five individual readings with a standard deviation not exceeding 0.3 mm per 20 m. The two height differences obtained at each setup could not differ by more than 0.3 mm or a full re-measure was undertaken. Despite temperature being observed at the terminals of each bay (section) by one of the field parties using a Kestrel digital weather tracker, observations were not corrected for temperature due to the relatively flat topography.

The field work presented several challenges (Figure 5). For example, heat shimmer forced the survey parties to include unscheduled breaks on several days, while wind caused significant concerns at other times with sighting distances being considerably shortened as a result. Backsight and foresight distances were kept equal (generally ±2 m) and varied between 5 m and 60 m.

Figure 5: Levelling field work in 2013.

In order to level across the Edward River at Moulamein, simultaneous and reciprocal observations were made at each side of the river (Figure 6). Five sets of height differences (B-F-F-B) were measured on each side of the river by both levelling parties and the mean height difference was taken. For all other creek and river crossings between Moulamein and Barham, conventional levelling techniques were adopted. This generally included one staff position on the bridge, placed at the location of a supporting pillar to minimise bridge movement during the observation. It was found that the resulting standard deviations were comparable to the remainder of the level run.

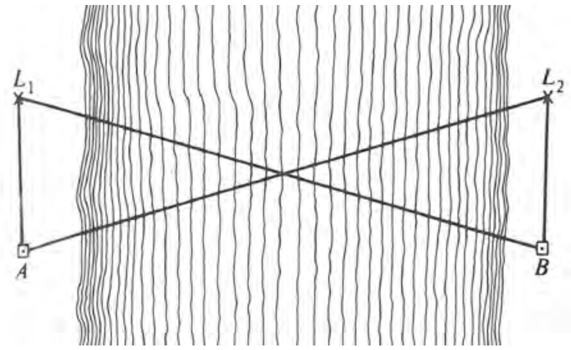

Figure 6: Simultaneous and reciprocal levelling across a river. L_1 and L_2 denote the instrument setups, while A and B denote the staff locations (Moffitt and Bouchard, 1982).

At the conclusion of each day's levelling, the misclose between the fore and back runs of each bay was checked to determine if it was within second-order specifications, i.e. $misclose < 8*\sqrt{k}$, where k is the distance of the level bay in kilometres and the misclose is obtained in millimetres (ICSM, 2007). All bays satisfied this limit and no re-measures were necessary. For a discussion of the terms Class and Order, the reader is referred to ICSM (2007) and Dickson (2012). While it is acknowledged that ICSM (2007) has recently been superseded by ICSM (2014), this update does not affect the outcome of this paper.

The levelling began at TS3479 (LCL3), 8 km north of Moulamein, and finished at PM22423 in Barham, totalling approximately 85 km. This levelling route coincides with part of two segments of the original Water Resources levelling: between SS16734 and TBM1043 and between TBM1120 and TBM1132. The levelling also incorporated a total of 41 state control marks, of which 14 had an existing vertical Class and Order of LDL4. In addition, a variety of existing benchmarks were observed, which were assigned Miscellaneous Mark numbers (28 marks in total).

Figure 7 illustrates the levelling route followed in this survey, between TS3479 and PM22423 (highlighted in blue). Also shown is the Water Resources levelling observed in 1979 (highlighted in green), which indicates where the two levelling routes overlap.

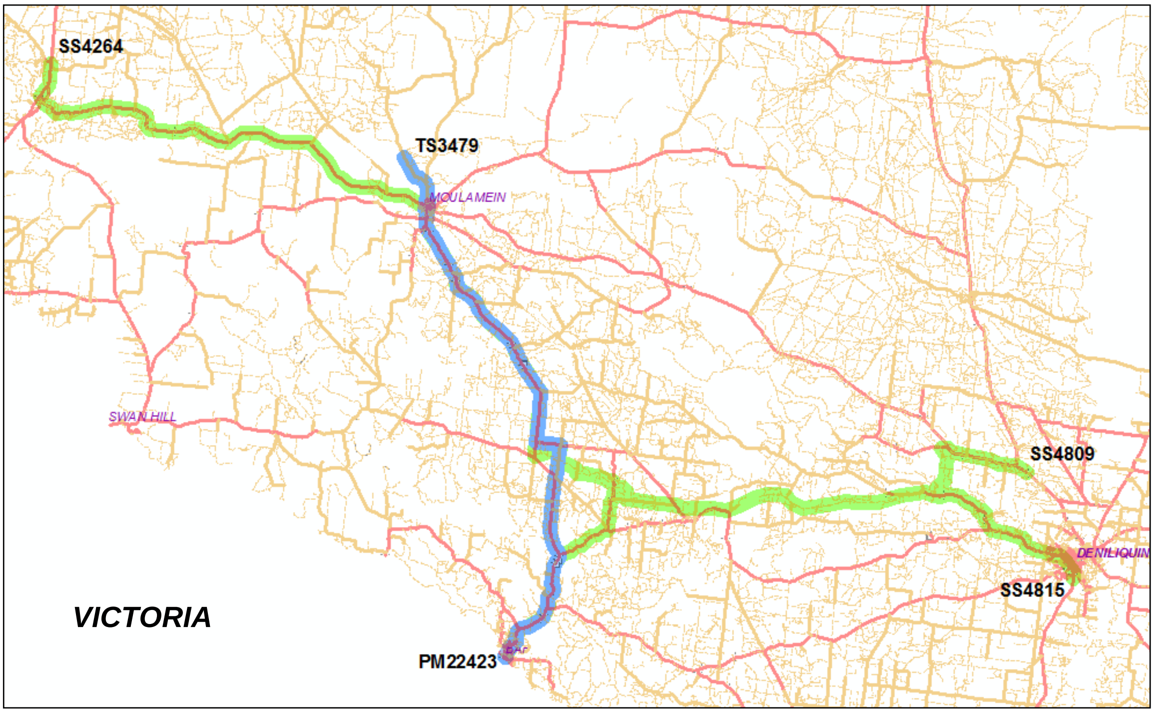

Figure 7: Levelling route taken in the 2013 survey (blue), and the original 1979 levelling (green).

4 ADJUSTMENT STRATEGY AND NETWORK GEOMETRY

The results from the second-order levelling connection between TS3479 and SS16734 in Moulamein again agreed to control to within 1 mm. Importantly, this section closed a large loop of existing third-order levelling: SS16734 – SS4264 – PM5809 – PM5808 – TS3479 – SS16734 (Figure 8). This closed loop of approximately 183 km had an overall misclose of 10 mm, comfortably meeting second-order specifications. This agreement within the loop further supported the basis of the datum for this adjustment.

The levelling network for this adjustment (see Figure 7) was simply a linear 85 km levelling route from Moulamein to Barham. The heights of two marks, TS3479 (LCL3) and SS16734 (LDL4), were constrained in the adjustment, while the other end of the levelling run in Barham was left unconstrained. This allowed us to establish a new datum in Barham to confirm either the NSW or Victorian levelling at the border.

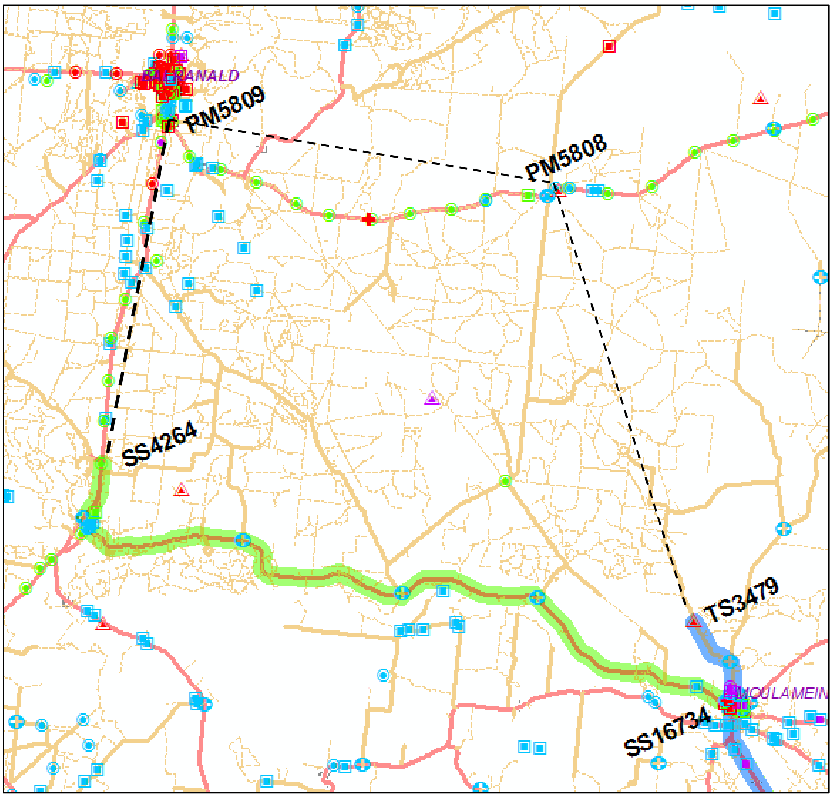

Figure 8: Closed levelling loop that provided the basis of datum for the levelling to Barham.

Figures 9 and 10 illustrate the levelling routes taken through Moulamein and Barham, respectively. As shown, a small number of existing state control marks were re-levelled in each town, providing a basis for future datum redefinitions in Moulamein and Barham.

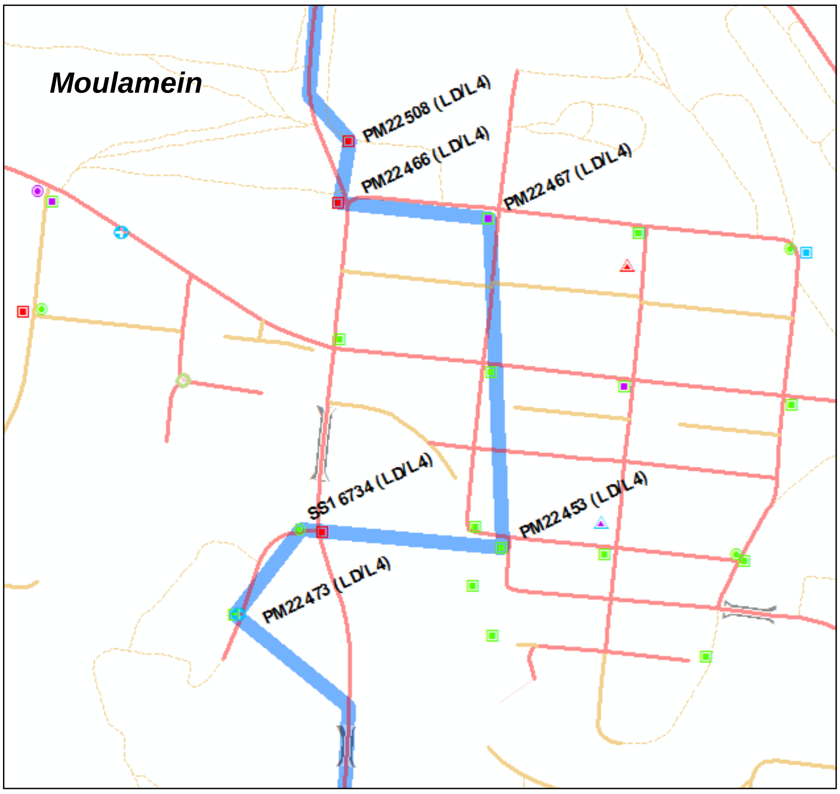

Figure 9: Levelling route through Moulamein (2013).

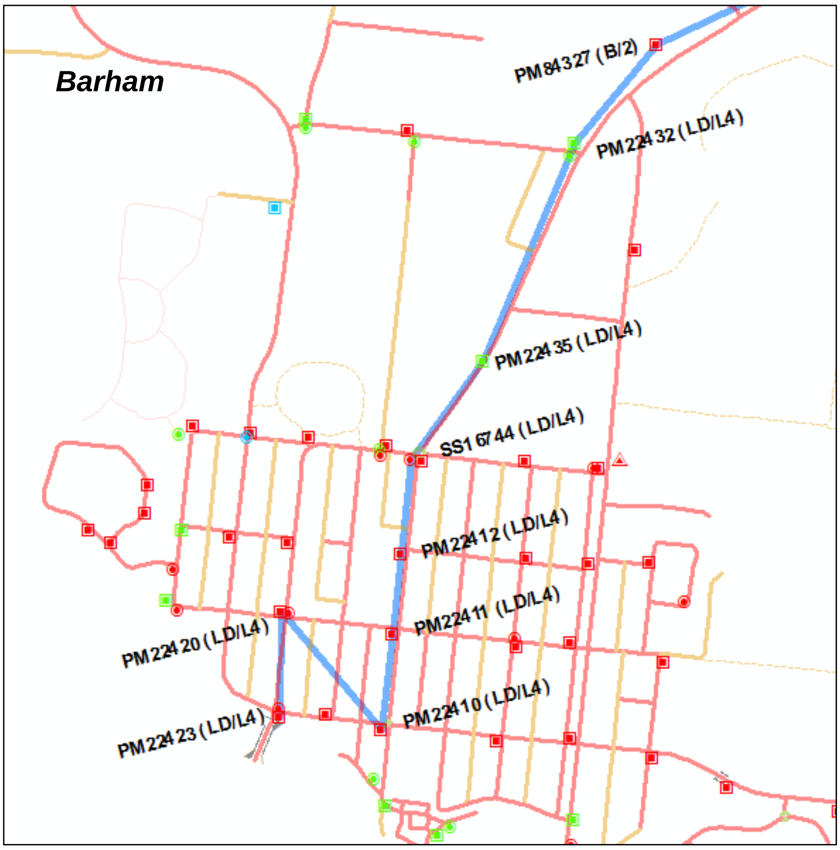

Figure 10: Levelling route through Barham (2013).

5 ADJUSTMENT RESUTS

Generally, a minimally constrained network adjustment is performed to evaluate the quality of the observations and check the data for outliers. This is then followed by a constrained adjustment in order to fit the new observations into the existing fabric and determine new or updated coordinates and/or heights for a number of marks to improve the control network.

5.1 Minimally Constrained Adjustment Results

As the network geometry of the levelling performed is linear, a minimally constrained adjustment is unnecessary because the residuals would all be zero. Instead, the quality of the levelling was assessed through the comparison between fore and back runs.

The overall misclose between fore and back runs was 6.1 mm over 85 km, which is substantially below the second-order allowable misclose of 73.9 mm. The maximum difference between any fore and back run was 3.4 mm in the 1.83 km bay between PM80590 and PM80591, which still comfortably met the second-order limit of 10.8 mm. The average difference between fore and back runs for each bay was 0.6 mm. This analysis, coupled with the measurement techniques adopted, confirmed unequivocally that the levelling met Class LB standards (ICSM, 2007).

5.2 Constrained Adjustment Results

Table 1 lists the two marks in Moulamein that were constrained in the final adjustment.

Table 1: Marks constrained in this adjustment and their vertical quality.

Mark	AHD (m)	Class	Order
TS3479	69.460	LC	L3
SS16734	71.391	LD	L4

As the levelling is linear and constrained at only one end, the adjustment statistics are of little relevance in this case. However, as this survey was conducted in an attempt to resolve a known discrepancy in the height datum between NSW and Victoria, an analysis of the results in Barham with respect to existing NSW and Victorian height information is necessary.

Firstly, the height comparisons to the original Water Resources benchmarks were analysed. A total of 19 Water Resources benchmarks were re-levelled, comprising 13 benchmarks cut into trees, 5 deep-driven rods (known as C-type benchmarks) and one pin in a concrete block (MM10327). Figure 11 provides examples of these mark types.

Figure 11: (a) Benchmark cut into a tree, (b) pin in concrete block (MM10327), and (c) C-type benchmark.

Agreement to within 25 mm was achieved at each of these 19 benchmarks with an absolute mean of 10.6 mm. This agreement was noticeably better on the C-type marks, with an absolute mean agreement of 7.8 mm, as well as MM10327 with an agreement of 1 mm. Considering the difference in mark types, along with the volatile nature of the soil topology in the area and the 34-year time lag between observations, these results are within an expected

range and therefore represent a reliable datum comparison to the original levelling performed in 1979.

Secondly, the newly derived heights of the marks in Barham (see Figure 10) were compared to their current SCIMS values. The new heights of the 8 LDL4 marks agreed with SCIMS to within 40 mm with an absolute mean of 17.9 mm. These results support the existing datum in Barham, considering the volatile nature of the soil and the age of the original levelling (mark movement is common in this area). However, it is important to note that all but one of these newly determined heights are above the current published heights. While this 'upward shift' in the datum in Barham brings it closer to the Victorian datum, a discrepancy of over 0.1 m still exists.

Based upon the survey methodology adopted, the existing survey control utilised and the mark types encountered, a vertical Class and Order can be assigned for each mark in this adjustment. Class LB was assigned for the 39 PMs and SSMs, as well as the C-type marks and for MM10327. All benchmarks in trees were downgraded to Class LD based on the marking quality. As the levelling was constrained to one third-order and one fourth-order mark in Moulamein, it was appropriate to assign a vertical Order of L4 to all marks in this adjustment.

6 DISCUSSION OF THE RESULTS IN NSW

As a result of this adjustment, the AHD heights of 67 marks (35 PMs, 4 SSMs and 28 MMs) were updated in SCIMS. Following the considerations outlined in section 5.2, 45 marks were assigned Class and Order LBL4, while the remaining 22 marks were assigned LDL4.

In particular, this adjustment provided new AHD heights for five state control marks in Moulamein and eight in Barham (see Figures 9 & 10). As a result, the AHD height of PM22423 in Barham was updated to 77.650 m in SCIMS, causing the height discrepancy at the NSW-Victoria border to decrease to 0.112 m at this mark. In order to homogenise the height datum within both Barham and Moulamein, it is suggested that levelling be conducted, either by Wakool Shire Council or a suitable contractor, to update the AHD heights of other marks not levelled in this survey. Following any such survey, it is requested that all relevant survey data be submitted to DFSI Spatial Services for update into the SCIMS database.

This survey had enough confidence instilled in the methods and results to suggest that the height error at Barham/Koondrook is not within the level section from Moulamein to Barham. The efforts taken to establish datum at the start of the level run in Moulamein, in conjunction with the standard deviations achieved throughout the survey as well as the methods involved, provide confidence in the correctness of this levelling section.

7 INVESTIGATIONS UNDERTAKEN IN VICTORIA

The results of the NSW investigations were forwarded to the Office of the Surveyor-General Victoria (OSGV), and it was recommended that additional work be directed towards the AHD anomaly from the Victorian side of the border. This section outlines the investigations undertaken to identify any possible contribution to the discrepancy within the Victorian network.

7.1 Levelling from Kerang to Barham/Koondrook

In 2006, OSGV surveyors conducted 2-way levelling, using third-order techniques, from Kerang to Barham/Koondrook (Figure 12). This level run established AHD heights on Victorian Survey Control Network (SCN) marks, spaced approximately every 1,600 m along the Kerang-Koondrook Road, and provided a direct level link from the Victorian Levelling Network (VLN) to these marks. This levelling survey included a level connection across the Murray River to PM22423 (1585-26) in Barham, which was the end point of the NSW levelling conducted in 2013 (see section 3.3).

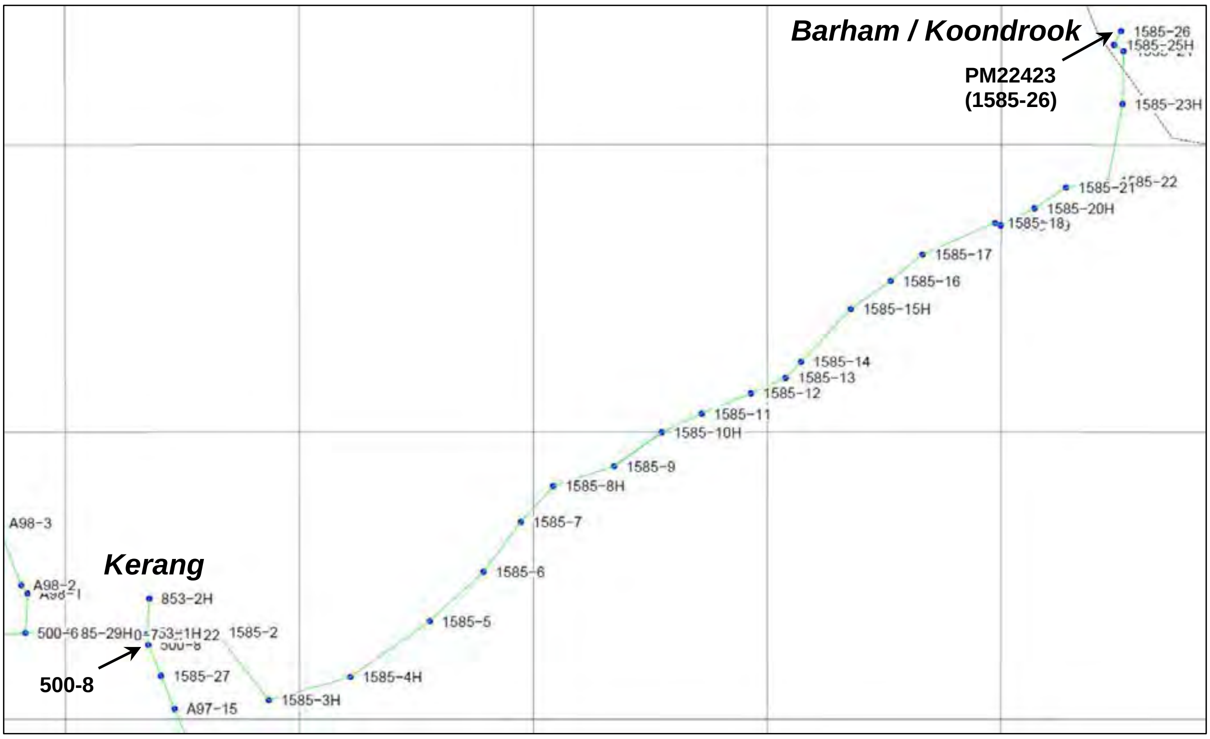

Figure 12: Victorian Levelling Network from Kerang to Barham/Koondrook.

In 2016, the 2006 Victorian level run from Kerang to Barham was re-investigated. The original level book (1585) was examined and it was confirmed that all the observed height differences in the levelling adjustment matched those observed in the field (allowing for scale factor corrections). As part of the 2006 level run, several marks in Kerang were check-levelled to confirm the datum, and it was noted that there was difficulty in confirming the datum in Kerang due to suspected mark instability in the area. It was decided to adopt the national junction point in Kerang (500-8) as the datum for the level run using the original 1968 value. Figure 13 illustrates the levelling around national junction point 500-8.

However, examination of the 2006 check-levelling observations indicated that the national junction point in Kerang (500-8) was the mark showing the most mark movement (notwithstanding some apparent movement in all marks). Marks 500-6 and 500-10 showed reasonable agreement and stability (6 mm), whereas the marks in between (i.e. 500-7, 500-8, A97-15) appear to have moved by 20-40 mm in height. A comparison of the levelling observations from 1968 and 2006 is shown in Table 2.

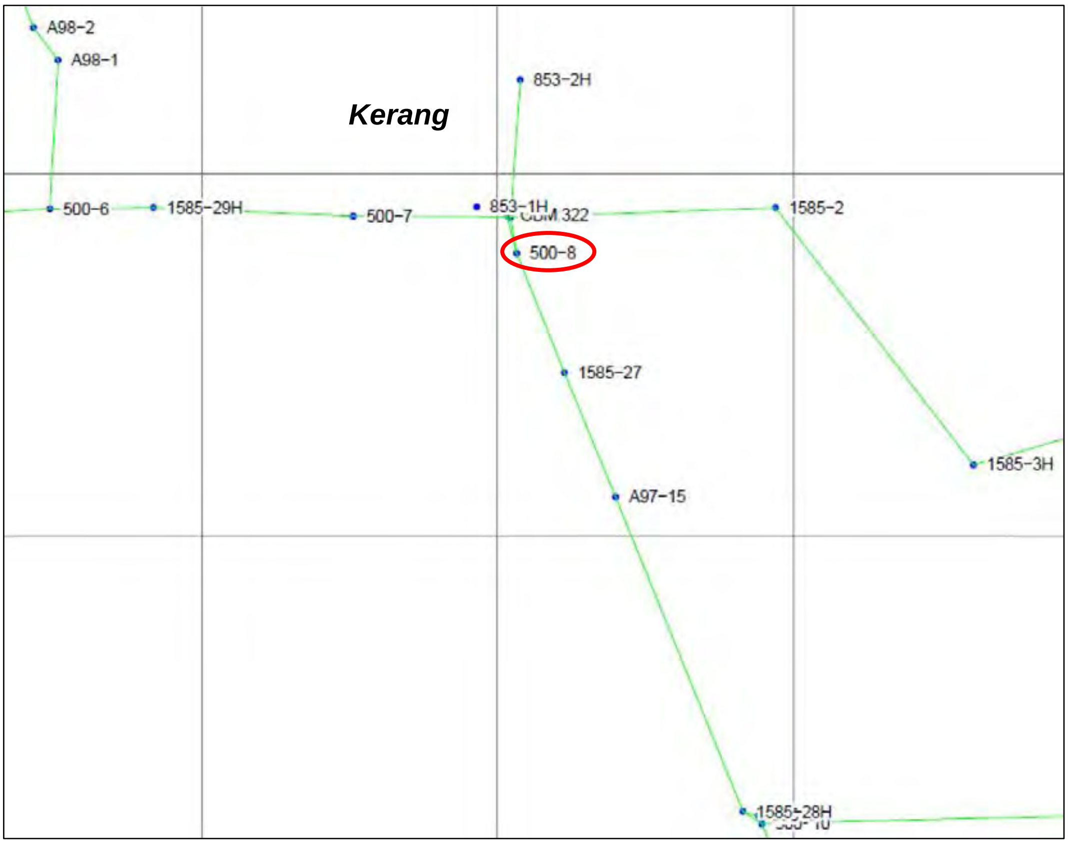

Figure 13: Victorian Levelling Network centred on Kerang with national junction point 500-8 highlighted.

Table 2: Comparison of levelled height differences observed in Kerang in 1968 and 2006.

From	To	ΔH_{1968} (m)	ΔH_{2006} (m)	Difference (m)
500-6	500-7	-0.375	-0.398	-0.023
500-7	500-8	0.348	0.331	-0.017
500-8	A97-15	0.205	0.232	0.027
A97-15	500-10	0.786	0.805	0.019

The changes seen in the height differences indicate that mark 500-8 had dropped by approximately 40 mm relative to marks 500-6 and 500-10. Therefore, it seems unwise to constrain the Koondrook level run to this potentially unstable mark, despite its status as a national junction point.

Victoria has a contiguous, state-wide levelling network (VLN) adjustment that contains all the levelling observations collected across Victoria by Geodetic Survey for the propagation of AHD across the State. The VLN adjustment includes all the level observations between marks along the level sections running between the junction points and making up the National Levelling Network (NLN). Additional levelling observations collected for the extension of AHD and the establishment of new AHD SCN marks are also included in the adjustment. The national junction points serve as constraint in the adjustment, with every junction point constrained to the AHD value derived from the NLN adjustment with an uncertainty of 5 mm. The assembly of all the levelling observations into one state-wide adjustment enabled the identification of various blunders in the original levelling observations.

The 2006 Kerang to Barham/Koondrook levelling observations are included in the VLN adjustment. Due to the nature of the adjustment, the level run is constrained to the national

junction point in Kerang (500-8), which is defined as the NLN-derived AHD height of 77.081 m.

In order to rectify the mark instability observed on mark 500-8 in the 2006 levelling, the VLN adjustment was re-run after removing the constraint on 500-8 and completely replacing the 1968 levelling observations to this mark with the measurements collected in 2006. In addition, the constraint on the High Stability Mark (HSM) in Koondrook (1585-25H – see Figure 12) to fix the adjustment at the border was also removed. This allowed the adjustment to derive a new height for mark 500-8 and subsequently all the marks along the level run from Kerang to Koondrook and Barham. Table 3 shows a comparison of the AHD heights derived from the original adjustment and the re-adjustment of the VLN with 500-8 and 1585-25H not constrained.

Table 3: Comparison of adjusted AHD values before and after removing the constraints in Kerang and Koondrook and removing old Kerang measurements to subsided mark (500-8) – * indicates a value that was constrained in the original adjustment.

Mark	Original Adj (m)	New Adj (m)	Difference (m)
500-6	71.110	77.110	0.000
500-7	76.734	76.710	-0.024
500-8	77.081 *	77.038	-0.043
500-10	78.076	78.070	-0.006
1585-25H	76.890 *	76.844	-0.046
1585-26 (PM22423)	77.762	77.716	-0.046

Removal of the constraint on the marks in Kerang and Koondrook, and update of the measurements in Kerang to those observed as part of the datum check in 2006, allows the adjustment to derive new AHD values for the marks out to Koondrook and Barham. Victoria's revised adjusted AHD height for PM22423 (1585-26) is now 77.716 m. When compared to the AHD height derived by DFSI Spatial Services in 2013 (77.650 m), this reduces the discrepancy to 0.066 m. Clearly, part of the anomaly was caused by combining modern levelling observations (2006) with the published AHD height of an old national junction point (500-8) that had potentially moved.

7.2 GNSS Validation from Kerang to Barham/Koondrook

In 2016, GNSS datasets were collected on four high-stability survey control marks in Kerang and Koondrook to confirm the levelling observations. Three to four hours of continuous static GNSS data was collected on two marks in Kerang (1585-1H and 1585-3H) and two marks in Koondrook (1585-23H and 1585-25H) (Figure 14).

All of these marks featured in the 2006 level run from Kerang to Barham/Koondrook. The GNSS data was submitted to Geoscience Australia's AUSPOS service (GA, 2017) to obtain high-precision ellipsoidal heights relative to GDA94. In order to overcome any potential bias to the AUSGeoid09 model (Brown et al., 2011) brought about by the inclusion of marks in Kerang and Koondrook, the gravimetric geoid model AGQG2009 (Featherstone et al., 2011) was used to interpolate the true geoid-ellipsoid separations in order to convert the ellipsoidal heights to orthometric heights.

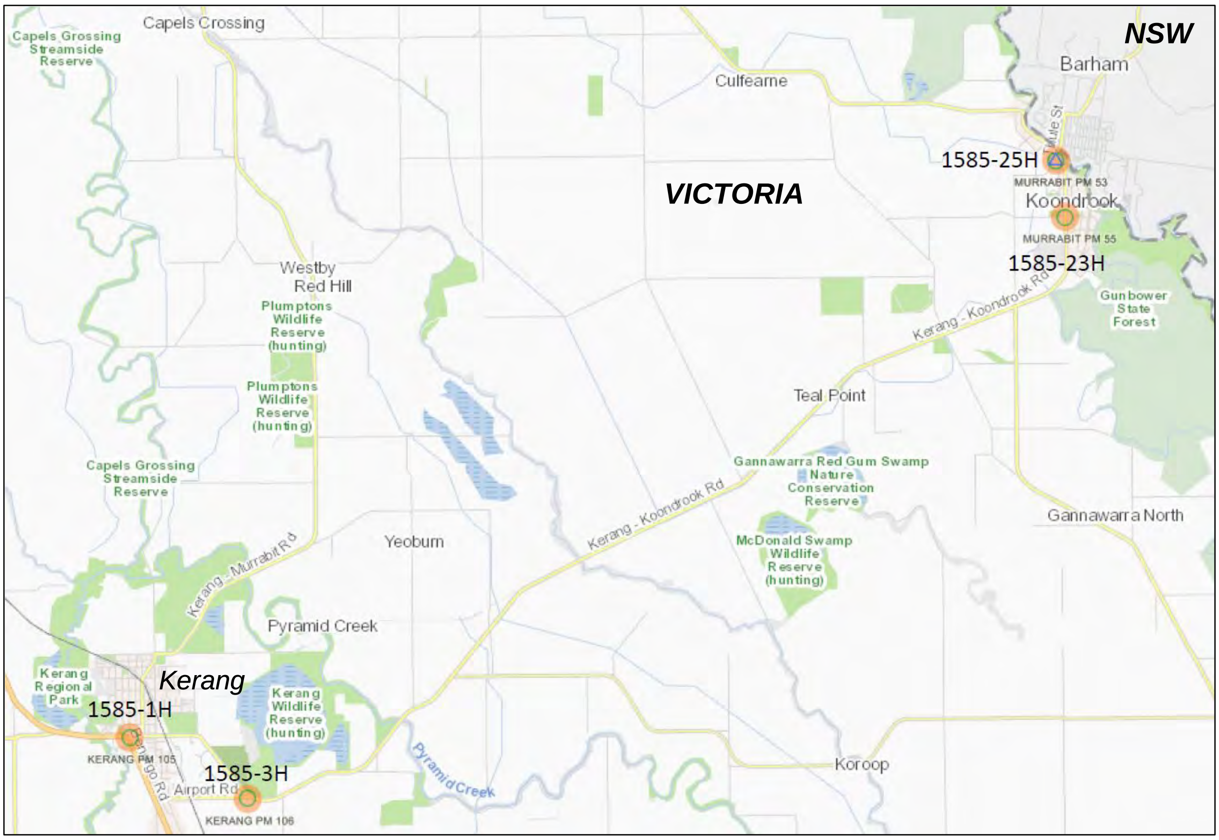

Figure 14: SCN marks observed in 2016 with GNSS in Kerang and Koondrook to verify the 2006 levelling observations.

Orthometric height differences (undistorted by any anomalies in AHD) were then determined between the marks and compared to the levelled height differences observed in 2006 (Table 4). The GNSS-derived height differences closely agree with the levelled height differences, indicating that there are no significant errors in the 2006 level run from Kerang to Barham/Koondrook.

Table 4: Comparison of height differences observed in the 2006 levelling and those observed using GNSS and the gravimetric geoid model in 2016.

From	To	$\Delta H_{Level(2006)}$ (m)	$\Delta H_{GNSS(2016)}$ (m)	Difference (m)
1585-1H	1585-3H	-0.890	-0.886	0.004
1585-3H	1585-23H	0.572	0.588	0.016
1585-23H	1585-25H	-0.362	-0.366	-0.004

7.3 Assessment of the National Levelling Network

The levelling and GNSS investigation carried out by DFSI Spatial Services from Moulamein to Barham confirmed that there were no apparent discrepancies in the NSW levelling data. Similarly, the levelling and GNSS investigation conducted by OSGV from Kerang to Barham/Koondrook provides no reason to doubt the levelling measurements in Victoria. If anything, there is an apparent error in the original 1966 NLN height for 500-8 of approximately 40-50 mm. Whether this is the result of subsidence or error in the original NLN remains unknown.

Based on the 2006 levelling and 2016 GNSS surveys and subsequent revision of the Victorian levelling adjustment (i.e. removal of the errant 500-8 constraint), it is concluded that the remaining 0.066 m AHD height discrepancy between the NSW and Victorian levelling observations, meeting at Barham, is attributed to the accumulation and distribution of error in the original 1966 levelling observations across the NLN.

The NLN consists of level sections running across the entire Australian continent. These level sections begin and end at junction points, often located at the intersection of major highways. Each level section is represented by a single height-difference observation from junction point to junction point, with an estimate of uncertainty. These height differences were obtained by combining the observed levelling height differences between survey marks along the level section and an uncertainty estimate derived based on the distance between junction points. The NLN was constrained to 30 tide gauges located around the coast of mainland Australia and two tide gauges along the coast of Tasmania, where MSL was fixed to 0.000 m. Figure 15 illustrates the NLN across Australia, while Figure 16 shows the NLN over the area of interest.

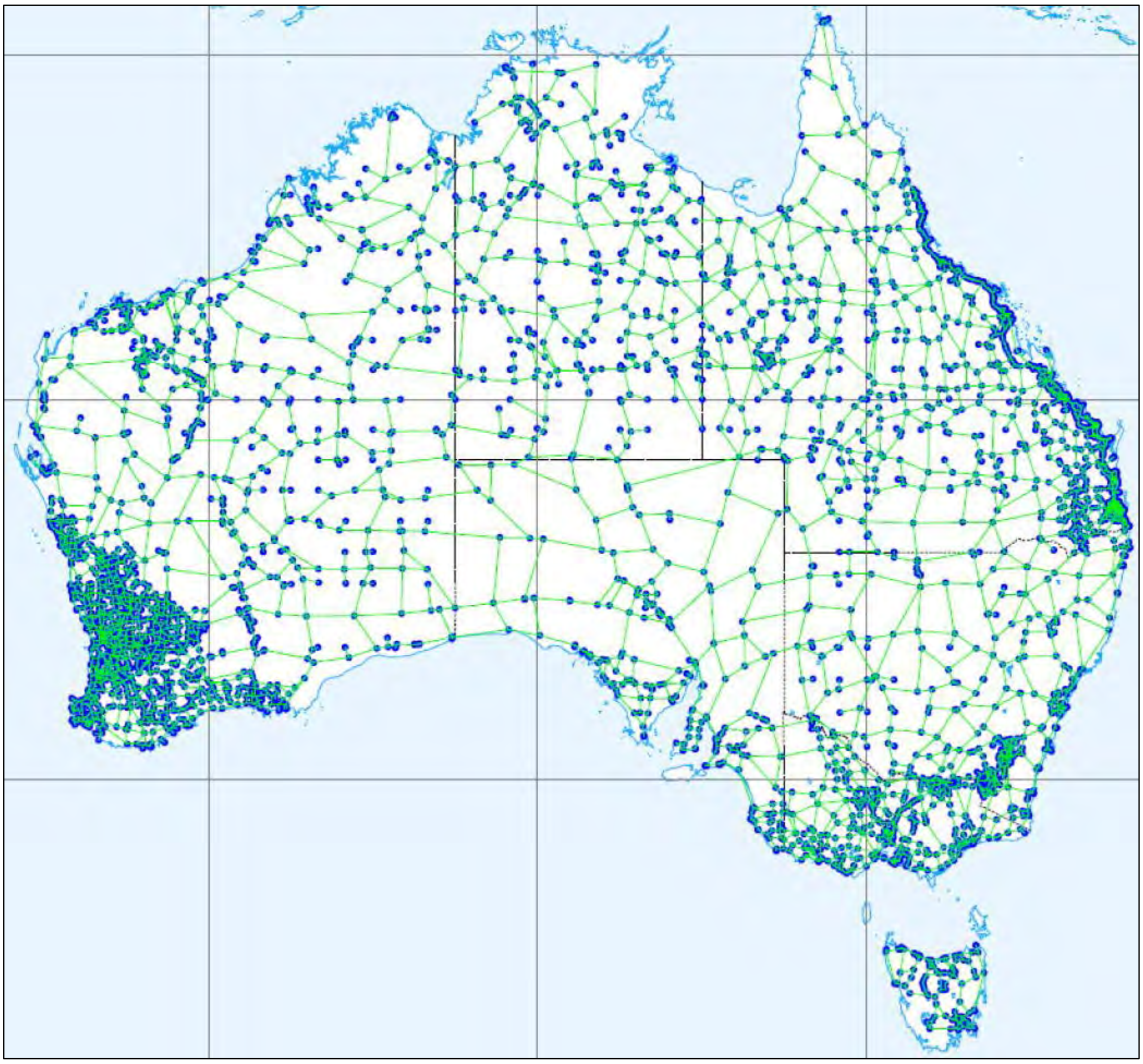

Figure 15: National Levelling Network across Australia.

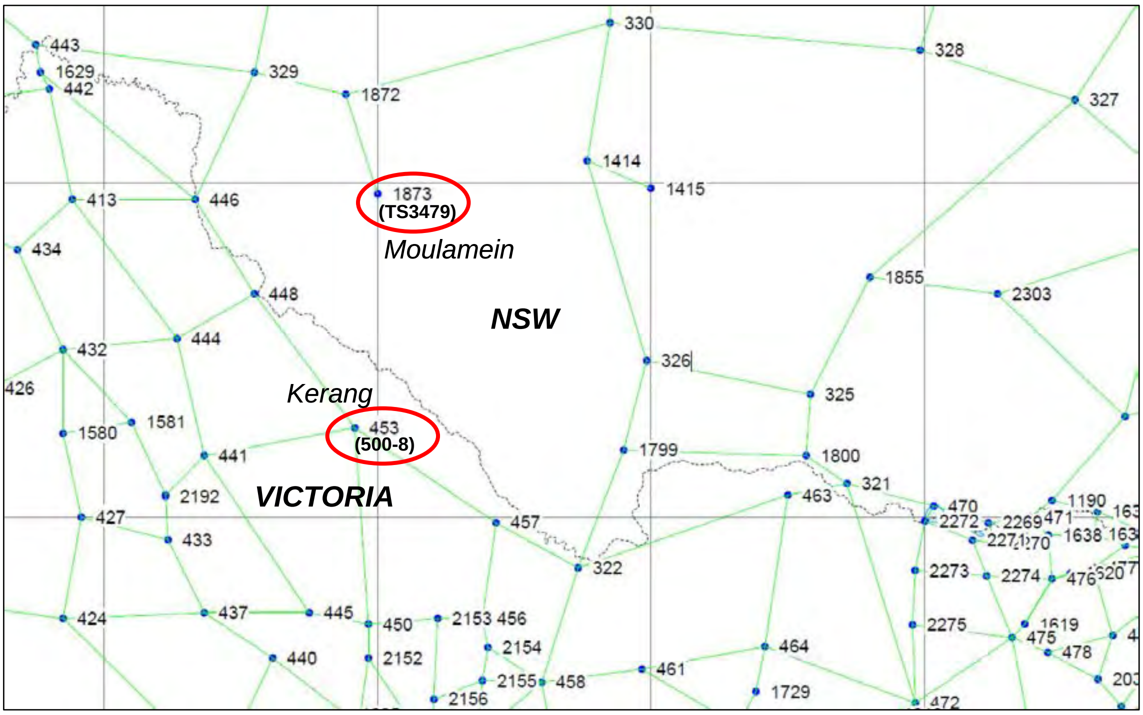

Figure 16: National Levelling Network centred over the NSW-Victoria border with the national junction points in Moulamein (NSW) and Kerang (VIC) highlighted.

As a result of the method applied for the AHD adjustment, any error in the NLN sections was distributed throughout the network. In order to investigate the spatial behaviour of the error distribution in the Barham/Koondrook area, an analysis of six individual level loops (obtained by forming a closed loop of individual level sections) was undertaken (Table 5).

Table 5: Level section loop misclose assessment – * indicates 453 is the junction point name for mark 500-8 in Kerang.

Loop Marks and Direction	Misclose (m)
322-457-453*-448-446-329-1872-330-1414-326-1799-322	-0.130
326-1414-330-328-327-1855-325-326	0.233
326-325-1800-1799-326	0.071
1799-1800-321-463-322-1799	0.092
453-457-456-2153-450-453	0.009
322-463-464-461-458-322	0.033

The brief assessment of the level loop miscloses shows the magnitude of error within the level sections used to create the NLN. The first loop misclose shown in Table 5 is the loop that incorporates Kerang (VIC) and Moulamein (NSW). The misclose of -0.130 m could account for the anomaly seen when the NSW level run from Moulamein to Barham is compared to the Victorian level run from Kerang to Barham/Koondrook. However, identifying and rectifying the source of this error could involve assessing hundreds of kilometres of levelling. The additional level loop miscloses indicate that the misclose can vary substantially across the network, with adjacent loops in NSW showing values of 0.071 m, 0.092 m and 0.233 m.

8 CONCLUDING REMARKS

It is well known that several anomalies exist in the Australian Height Datum. This paper has outlined investigations conducted collaboratively by DFSI Spatial Services and the Office of

the Surveyor-General Victoria into one such anomaly, which initially amounted to approximately 0.14 m and occurs at the NSW-Victoria border between Barham and Koondrook. Understandably, this discrepancy has major implications for surveyors and spatial professionals working on either side of the state border, as well as for flood management of the Murray River. Recent survey work performed by DFSI Spatial Services utilised GNSS technology and 85 km of conventional, second-order, 2-way levelling conducted in 2013 to identify the source of the anomaly. It was found that the levelling connections are consistent. Based on the levelling conducted in 2013, the AHD height of PM22423 in Barham was updated to 77.650 m, causing the height discrepancy between NSW and Victoria to decrease to 0.112 m at this mark.

Examining third-order levelling data collected in 2006 and GNSS data observed in 2016, the Office of the Surveyor-General Victoria revealed that issues arise when National Levelling Network (NLN) junction points are constrained and new levelling observations are taken between junction points. In this case, the Moulamein junction point (1873 or TS3479) was constrained at 69.460 m and AHD propagated along the level run to Barham, and the Kerang junction point (453 or 500-8) was constrained at 77.081 m and AHD propagated along the level run to Barham. NSW and Victoria are both confident in the quality of these level runs, with GNSS observations confirming height differences. The Office of the Surveyor-General Victoria modified the constraint in Kerang to better fit the 2006 levelling and to allow for the suspected mark instability at the junction point. This accounts for 0.046 m of the discrepancy at Barham. Therefore, the remaining 0.066 m discrepancy is most likely located within the NLN – and is most likely present in both NSW and Victoria. Due to the nature of the original AHD levelling adjustment, it is difficult to identify the precise location of the error source because generally it is distributed across many level sections throughout the network.

In order to rectify this problem in a conclusive way, the NSW and Victorian level run data will be combined into a contiguous adjustment, linking the junction points in Moulamein (1873) and Kerang (453). This will allow the 0.066 m discrepancy in the NLN to be distributed across the entire level run, and ensure much closer harmony between AHD heights on either side of the Murray River in Barham/Koondrook.

This paper is also a showcase example of collaboration between the state government departments responsible for maintaining survey control across NSW and Victoria. Together, DFSI Spatial Services and the Office of the Surveyor-General Victoria have identified, investigated and solved (at least in part) an AHD anomaly at the NSW-Victoria border. The outcome of this investigation will not only benefit surveyors and spatial professionals working on either side of the state border but also improve flood management of the Murray River.

ACKNOWLEDGEMENTS

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Performance of AUSGeoid09 in Mountainous Regions

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ABSTRACT

The Australian Height Datum (AHD) is our current national vertical datum, and AUSGeoid09 is the current quasigeoid model used to compute (normal-orthometric) AHD heights from Global Navigation Satellite System (GNSS) derived ellipsoidal heights given in the Geocentric Datum of Australia 1994 (GDA94). While previous studies have evaluated the AUSGeoid09 model across Australia, these have generally not focused on mountainous regions in particular. This paper investigates the performance of AUSGeoid09 in the Mid Hunter and Snowy Mountains regions of New South Wales, from a user's perspective. Comparisons are undertaken in an absolute sense (i.e. single point) and relative sense (i.e. height difference between two points) between AUSGeoid09-derived heights and official AHD heights on public record. The performance of AUSGeoid09 is evaluated relative to its predecessor AUSGeoid98. In both study areas, an overall improvement is evident when applying AUSGeoid09 to compute AHD heights. However, a slope was detected for AUSGeoid09 residuals in the Snowy Mountains, and it appears that the geometric component may have overcompensated for sea surface topography in this area. AUSGeoid09 generally provided AHD height differences at the ±0.05 m to ±0.09 m level (1 sigma) and substantially increased the percentage of GNSS-derived height differences meeting third-order differential levelling specifications. This is a very encouraging result, considering the difficulties of spirit levelling in mountainous terrain and the increasing popularity of GNSS-based height transfer in practice. However, it has been shown that some discrepancies still remain between AUSGeoid09-derived heights and AHD. It is anticipated that the new AUSGeoid2020 (to be used in conjunction with GDA2020 ellipsoidal heights) will substantially improve access to AHD via GNSS techniques. The methodology presented in this paper will be very beneficial in regards to future testing of AUSGeoid2020.

KEYWORDS: *AUSGeoid09, AHD, geoid model, N values, GNSS, national datums.*

1 INTRODUCTION

The Geocentric Datum of Australia 1994 (GDA94) has been our national datum since its adoption in 2000, providing fundamental geodetic infrastructure for Australia (ICSM, 2014a).

It should be noted that a new, much improved Australian national datum, GDA2020, will be released this year and is planned to be adopted by 1 January 2020 (e.g. ICSM, 2016; Gowans, 2017; Janssen, 2017).

The Australian Height Datum (AHD) is our first and only national vertical datum. It was defined by setting to zero the average Mean Sea Level (MSL) values of 32 tide gauges around Australia for a period of about two years that began in 1966 and adjusting 97,230 km of 2-way spirit levelling (Roelse et al., 1975). Now, 50 years later, it is well known that shortcomings in the AHD realisation (AHD71 for mainland Australia and AHD83 for Tasmania) resulted in MSL not being coincident with the geoid at the tide gauges involved.

These shortcomings included not considering dynamic ocean effects (e.g. winds, currents, atmospheric pressure, temperature and salinity), a lack of long-term tide gauge data, and the omission of observed gravity. This has introduced considerable distortions of up to about 1.5 m into AHD across Australia, which is therefore considered a third-order datum (e.g. Morgan, 1992; Featherstone and Filmer, 2012; Watkins et al., 2017). However, AHD continues to be a practical height datum that provides a sufficient approximation of the geoid for many surveying and engineering applications. Consequently, in practice, AHD heights are often accepted as being equivalent to orthometric heights. For a review of coordinate systems, datums and associated transformations in the Australian context the reader is referred to Janssen (2009a, 2009b). A review of Australian height systems and vertical datums can be found in Featherstone and Kuhn (2006).

Over the last two decades, Global Navigation Satellite System (GNSS) technology has become the primary positioning tool due to its accuracy, speed and accessibility. GNSS-based heights refer to a reference ellipsoid, i.e. a purely mathematical representation of the earth, and therefore have no physical meaning. In most practice, however, heights are required that correctly reflect the flow of fluids, e.g. for flood modelling, drainage and pipeline design. Hence, a reliable geoid model is required to derive AHD heights from measured ellipsoidal heights.

N values (N), also known as geoid undulations or geoid-ellipsoid separations, can be used to convert GNSS-derived ellipsoidal heights (h) to AHD heights (H) and vice versa (provided N and h refer to the same ellipsoid):

$$H = h - N \tag{1}$$

It is worth noting that H is often used to indicate orthometric heights (i.e. relative to the geoid), while Reduced Level (RL) refers to AHD heights. However, for the sake of simplicity and considering that in surveying and engineering practice both are often assumed the same, we use H to indicate AHD heights in this paper.

For many years, the use of geoid models – or quasigeoid models, see e.g. Vaniček et al. (2012) and Sjöberg (2013) for a discussion of the difference – has helped GNSS users to compute AHD heights from ellipsoidal heights. In the Australian context, AUSGeoid09 is the latest quasigeoid model to be used in conjunction with GDA94 ellipsoidal heights (Brown et al., 2011; Featherstone et al., 2011). While it should be noted that a new version, AUSGeoid2020, will soon be released for exclusive use with GDA2020 ellipsoidal heights (e.g. ICSM, 2016; Gowans, 2017; Janssen, 2017), this paper focuses on AUSGeoid09.

The performance of AUSGeoid09, along with the improvements it provides over its predecessor AUSGeoid98, has been investigated previously (e.g. Janssen and Watson, 2010, 2011; Brown et al., 2011). However, very few studies have focused on mountainous regions. Considering that gravity can change dramatically within a few kilometres on the earth's surface in Australia (Darbeheshti and Featherstone, 2009), especially in mountainous terrain, and that observed gravity data are generally sparse in these areas, it is necessary to evaluate the performance of AUSGeoid09 in mountainous regions in particular.

Geoid or quasigeoid models are commonly verified by using GNSS and orthometric height data. This can be done in an absolute and relative sense (Featherstone, 2001): An absolute verification estimates the accuracy and precision of the (quasi)geoid, with respect to the geocentric ellipsoid, using GNSS networks that have been tied to an (inter)national reference frame and spirit-levelled orthometric heights that have been tied to the (national) vertical datum. A relative verification utilises GNSS-derived ellipsoidal height differences and spirit-levelled orthometric height differences to estimate the accuracy and precision of the (quasi)geoid gradients.

This paper evaluates the performance of AUSGeoid09 in the Mid Hunter and Snowy Mountains regions of New South Wales (NSW), from a user's perspective. Comparisons are undertaken in an absolute sense (i.e. single point) and relative sense (i.e. height difference between two points) between AUSGeoid09-derived heights and official AHD heights on public record in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013). The performance of AUSGeoid09 is evaluated relative to its predecessor AUSGeoid98. It should be noted that this paper combines the main findings of the absolute and relative investigations previously reported in two separate journal papers (Sussanna et al., 2014, 2016).

2 AUSGeoid09

AUSGeoid09 was released in March 2011 by Geoscience Australia, replacing the previous model AUSGeoid98 (Featherstone and Guo, 2001). Both models refer to the GRS80 ellipsoid, which was adopted as the reference ellipsoid for GDA94, and cover the same geographical area between 108ºE and 160ºE longitude and between 8ºS and 46ºS latitude. However, AUSGeoid09 is provided as a 1' by 1' grid (approximately 1.8 by 1.8 km), making it four times denser than its predecessor (Featherstone et al., 2011).

Previous versions of AUSGeoid were predominantly gravimetric-only quasigeoids, and it was assumed that these were sufficiently close approximations of AHD – an assumption we now know to be incorrect because AHD is not as closely aligned to the geoid as we would like. In contrast, AUSGeoid09 is a combined gravimetric-geometric quasigeoid, providing a direct connection to AHD and thereby allowing a more reliable determination of AHD heights from GNSS observations (Brown et al., 2011). The empirically derived geometric component accounts for the offset between the gravimetric quasigeoid and AHD, which is predominantly caused by AHD not taking into account sea surface topography including the differential heating of the oceans.

Since the warmer or less dense water off northern Australia is about 1 metre higher than the cooler or denser water off southern Australia, AHD is about 0.5 m above the quasigeoid in northern Australia and roughly 0.5 m below the quasigeoid in southern Australia (Brown et

al., 2011; Janssen and Watson, 2010, 2011). The introduction of the geometric component takes care of most of this 1-metre trend across Australia (0.6-metre trend across NSW), thereby providing a better overall fit to AHD.

AUSGeoid09 has been shown to convert ellipsoidal heights to AHD heights with an accuracy of ±0.03 m (1 sigma), or ±0.06 m at the 95% confidence interval (CI), with the exception of some pockets where the misfit can be larger than ±0.1 m (1 sigma) or ±0.2 m (95% CI) due to errors caused by factors such as the ageing levelling network, geoid height variability or data deficiency (Brown et al., 2011). Using a more practical approach, Janssen and Watson (2010, 2011) found that AUSGeoid09 generally allows GNSS-based height determination in NSW at the ±0.05 m level (1 sigma) or ±0.10 m (95% CI). In contrast, its predecessor AUSGeoid98 only provides an absolute accuracy of ±0.4 m (Featherstone and Guo, 2001; Featherstone et al., 2001).

Using Network Real Time Kinematic (NRTK) GNSS observations in the Blue Mountains area of NSW, Allerton et al. (2015) found that AUSGeoid09 allows AHD height determination at the ±0.03 m level (1 sigma) or ±0.06 m (95% CI) in flat terrain and at the ±0.06 m level (1 sigma) or ±0.12 m (95% CI) in mountainous terrain. However, results also indicate that there is room for improvement in regards to future versions of the AUSGeoid model for elevations above 500 m. In this paper, we use a much larger dataset to investigate AUSGeoid09 performance in mountainous areas.

3 ABSOLUTE AND RELATIVE GNSS HEIGHTING

In an absolute sense, also known as the single point approach, the GNSS-derived ellipsoidal height at a point can be converted to a (normal-orthometric) AHD height using equation (1). This is generally applied when users connect to a GNSS Continuously Operating Reference Station (CORS) network such as CORSnet-NSW (Janssen et al., 2016; DFSI Spatial Services, 2017), e.g. via NRTK observations, or use Precise Point Positioning (PPP) to derive AHD heights.

In a relative sense, the AHD height of a point B is calculated relative to another point A with known AHD height, using the differences (Δ) in GNSS-derived ellipsoidal heights and N values supplied by the geoid model:

$$H_B = H_A + \Delta H_{AB} \quad \text{with} \quad \Delta H_{AB} = \Delta h_{AB} - \Delta N_{AB} \qquad (2)$$

The well-known advantage of the relative method is that simultaneous observations at both points minimise most of the systematic errors by virtue of the difference, as is the case with GNSS baseline processing. The absolute N values at both points may have relatively large errors, but the height of point B is only contaminated by the small difference of these errors (ignoring any GNSS observational errors). Figure 1 illustrates these two approaches.

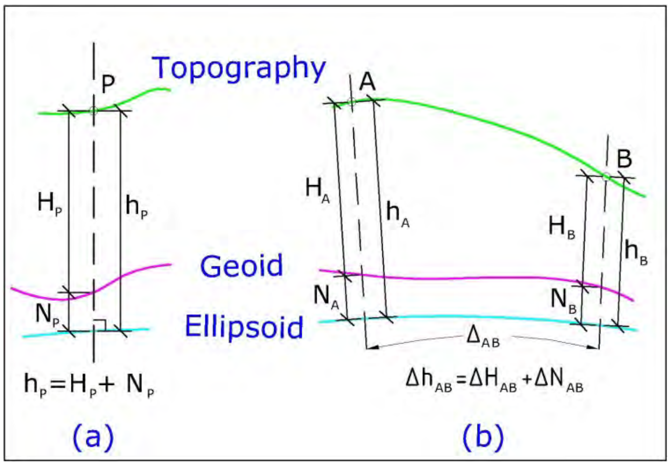

Figure 1: Relationship between ellipsoidal height (h), normal-orthometric AHD height (H) and geoid-ellipsoid separation (N) for (a) absolute and (b) relative GNSS heighting.

4 METHODOLOGY: ABSOLUTE AND RELATIVE GEOID MODEL VERIFICATION

4.1 Absolute Verification

Due to the increased use of CORS networks, the absolute accuracy of N values is now more important than ever for GNSS-based AHD height determination (Janssen and Watson, 2010, 2011). In an absolute sense, AUSGeoid09 performance was verified based on the comparison of a network of GNSS observations and official AHD heights published in SCIMS. Using equation (1), AHD-derived N values (N_{AHD}) were computed by subtracting the published AHD height (H_{AHD}) at each checkpoint from the ellipsoidal height (h) obtained from a least squares network adjustment using the GeoLab software (BitWise Ideas, 2017).

These N_{AHD} values were compared with N values computed using AUSGeoid09 (and AUSGeoid98) to determine residuals ($N_{AG} - N_{AHD}$) for the following two tests, both statistically and graphically (Featherstone and Guo, 2001):

- Comparison for all checkpoints.
- Comparison as a function of AHD height.

Descriptive statistics were computed to obtain a numerical representation of the population sample, as previously adopted by Brown et al. (2011). Z-statistics were employed to identify any outliers, in this paper defined as three times larger than the standard deviation. Since it is necessary to consider residuals of different signs, the Root Mean Square (RMS) was also utilised.

Generally speaking, the primary aim of the absolute verification was to quantify the accuracy of AUSGeoid09 in regards to computing AHD heights at single points in mountainous regions. Furthermore, a comparison between AUSGeoid09 and its predecessor AUSGeoid98 was performed to quantify the expected improvement in mountainous areas.

4.2 Relative Verification

In spite of the growing popularity of CORS networks and PPP for GNSS-based height transfer, surveyors often continue to use the relative GNSS heighting method to 'carry' heights from established marks to unestablished marks via baseline processing. As indicated

earlier, this concept is less affected by errors in the N values since common systematic errors are minimised by virtue of differencing.

GNSS baselines were observed between points to obtain the differences in ellipsoidal heights, which were then converted into AHD height differences using the computed N values. This can be applied over all possible mark-to-mark vector combinations within a network of control points. However, a least squares adjustment is required to create a network with consistent ellipsoidal heights. The number of possible vector combinations is $n(n\text{-}1)/2$ where n denotes the number of points.

AUSGeoid09 and AUSGeoid98 performance was investigated by comparing between any two points the difference in GNSS-derived AHD heights (ΔH_{GNSS}) computed using equation (2) with the difference in official AHD heights (ΔH_{AHD}) for these two points. Residuals (R) were then computed as follows:

$$R = \Delta H_{AB(GNSS)} - \Delta H_{AB(AHD)} \tag{3}$$

The following four tests were performed, both statistically and graphically, and analysed based on descriptive statistics:

- Comparison for all observed baselines.
- Comparison for all possible mark-to-mark combinations.
- Comparison for all possible mark-to-mark combinations up to 100 km in length.
- Comparison for all possible mark-to-mark combinations as a function of AHD height difference.

The primary aim of the relative verification was to quantify the accuracy of AUSGeoid09 in regards to computing AHD height differences in mountainous terrain. A comparison between AUSGeoid09 and its predecessor AUSGeoid98 was performed to quantify the expected improvement in mountainous areas. Furthermore, the baseline residuals were compared to the allowable misclose of third-order differential levelling (ICSM, 2007), i.e. the maximum allowable misclose of $12\sqrt{d}$ (d = distance in km, here calculated on the GRS80 ellipsoid using Vincenty's inverse formula). Residuals were also expressed as parts per million (ppm).

5 STUDY AREAS AND DATASETS

The performance of AUSGeoid09 in mountainous regions was evaluated in two study areas located in NSW (Figure 2). Both study areas represent typical mountainous terrain conditions encountered in Australia and exhibit large differences in elevation. Spatial Services, a unit of the NSW Department of Finance, Services and Innovation (DFSI), provided two GNSS network datasets collected over many years, together encompassing 186 survey marks with known AHD heights of sufficient quality (Class C Order 3 or better) on public record in SCIMS. For a discussion of the terms class and order, the reader is referred to ICSM (2007) and Dickson (2012). While it is acknowledged that ICSM (2007) has recently been superseded by ICSM (2014b), this update does not affect the outcome of the analysis presented here.

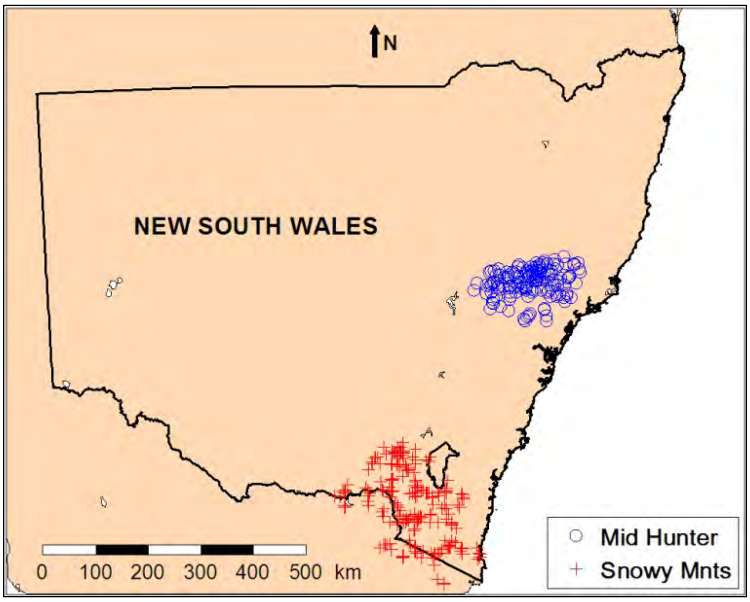

Figure 2: Location of the two study areas in NSW.

The Mid Hunter GNSS network adjustment covers an area of approximately 13,000 km^2, stretching from about 115 km south of the Mount Royal National Park to 170 km east of Mudgee. The terrain is mainly composed of valleys and mountains with elevations ranging between 20 m and 1,400 m. This dataset consists of 327 independent GNSS baselines observed between 147 marks. Of these, 82 SCIMS marks have known AHD heights (C3 or better), including 40 spirit-levelled marks of classification LCL3 or better.

The Snowy Mountains GNSS network adjustment covers an area of about 35,000 km^2, approximately bounded in the north by Tumut and the ACT border to Cooma, and in the south by Albury and the Victorian border towards the coast. The terrain exhibits an undulated topography composed of mountains reaching a peak of 2,228 m (Mt Kosciuszko) and low valleys with elevations of about 200 m. The GNSS dataset consists of 629 independent baselines observed between 263 marks. Of these, 104 SCIMS marks have known AHD heights (C3 or better), including 94 spirit-levelled marks of classification LCL3 or better.

In total, across both study areas, this provided 186 checkpoints with known AHD heights of sufficient quality for a practical AUSGeoid09 performance verification and a comparison to AUSGeoid98. It should be noted that while some of the GNSS data used in this study contributed to the generation of AUSGeoid09, the datasets are considered sufficiently independent for the purpose of this study.

6 DATA PROCESSING AND ANALYSIS

The two GNSS networks used in this study were subject to several adjustments performed using the GeoLab least squares adjustment software (BitWise Ideas, 2017). These adjustments were constrained to the national datum (GDA94) via several AUSPOS solutions (GA, 2017), i.e. 7 and 27 marks in the Mid Hunter and Snowy Mountains networks respectively. N values were computed based on bi-cubic interpolation, using both AUSGeoid09 and AUSGeoid98 to enable comparison between the two models. The resulting GNSS-derived AHD heights are therefore independent of the official AHD heights on public record.

Both the Mid Hunter and the Snowy Mountains networks generated Class A surveys as per ICSM (2007). The ellipsoidal heights for the 104 checkpoints in the Snowy Mountains network displayed an average uncertainty of ±0.016 m (1 sigma) or ±0.031 m (95% CI). The Mid Hunter network performed slightly better, resulting in an average uncertainty of ±0.012 m (1 sigma) or ±0.024 m (95% CI) for the 82 checkpoints.

6.1 Absolute Verification

For absolute verification, N_{AHD} values were computed at each checkpoint from the ellipsoidal heights obtained via the constrained least squares adjustment and published AHD heights at these survey marks using equation (1). On the same checkpoints, N values (N_{AG}) were also obtained from AUSGeoid09 and AUSGeoid98, and then compared to their N_{AHD} values.

6.1.1 Absolute Comparison for all Checkpoints

In order to examine the distribution of the residuals ($N_{AG} - N_{AHD}$), a Kurtosis test was performed. A negative Kurtosis value indicates flatness with a large number of residuals concentrated along the side of a normal distribution, while a positive value denotes a sample of a peak with the majority of the residuals concentrated in the proximity of the mean (DeCarlo, 1997). It was found that the AUSGeoid09 residuals in both study areas are more consistent with a normal distribution with a large amount of residuals close to the mean, while AUSGeoid98 residuals denoted flatness with a large amount of residuals along the side of the normal distribution.

The comparison of the two quasigeoid models in the Mid Hunter study area shows a substantial improvement using AUSGeoid09 over its predecessor, evidenced by the standard deviation dropping from ±0.074 m to ±0.040 m. The RMS for AUSGeoid09 indicates an improvement factor of 6 compared to AUSGeoid98. Neither of the two models indicated any outliers of three times their standard deviation.

As evident from Figure 3, the AUSGeoid98 residuals are heavily positive at around 0.25 m, while the AUSGeoid09 residuals are well distributed around a near-zero mean. AUSGeoid09 provides an exceptional improvement over AUSGeoid98, showing no trends and exhibiting consistent residuals across the study area.

Figure 3a suggests the existence of a slope in AUSGeoid98 residuals in the longitudinal direction, dipping towards the east of the network, while Figure 3b indicates a rise in the middle of the study area's latitudinal extent. The heavily positive AUSGeoid98 residuals are consistent with the geometric component of AUSGeoid09 generally amounting to about -0.2 m in this area (Brown et al., 2011), clearly showing the beneficial effect the introduction of the geometric component has on GNSS-derived AHD height determination in this area.

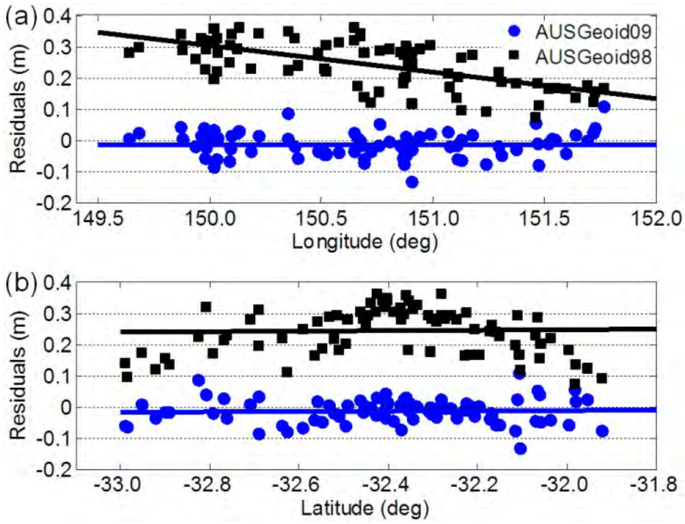

Figure 3: Mid Hunter verification: Absolute residuals between quasigeoid-derived (AUSGeoid09 and AUSGeoid98) N values and N_{AHD} values for (a) increasing longitudes and (b) decreasing southern latitudes.

The comparison of the two quasigeoid models in the Snowy Mountains study area reveals a moderate improvement of AUSGeoid09 over AUSGeoid98 with the standard deviation dropping from ±0.090 m to ±0.070 m. However, no improvement was detected in the RMS. While neither model showed any outliers greater than three times the standard deviations, the mean of the residuals is closer to zero for AUSGeoid98 than AUSGeoid09 because the AUSGeoid98 residuals are almost equally balanced between positive and negative values ranging between -0.224 m and 0.165 m. In contrast, the majority of AUSGeoid09 residuals are negative ranging from -0.284 m to 0.059 m.

Figure 4a illustrates the residuals of both quasigeoid models as a function of their longitudinal position. It is evident that AUSGeoid98 residuals show a relatively large scatter but no trend. On the other hand, AUSGeoid09 residuals exhibit a lesser spread but show a trend where the residuals seem to increase (larger negative values) in the eastern direction.

Both models show a similar trend as a function of the latitudinal position, with residuals decreasing (smaller negative values) in the northern direction (Figure 4b). A closer evaluation of the residuals identifies that AUSGeoid98 residuals are of larger magnitude and larger spread from the mean, while AUSGeoid09 residuals are more closely aligned with the mean, the only exception being two checkpoints at latitude -37.0º (and longitude 149.9º). If these two residuals are removed, the range in AUSGeoid09 residuals decreases by about 75 mm and the standard deviation improves to ±0.065 m. However, there is not enough evidence supporting a gross error within these two checkpoints to justify their removal.

Figure 4 shows that a slope is evident within AUSGeoid09 residuals from the north-west to the south-east corner, with the majority of negative residuals situated at the south-east corner of the study area. This direction is perpendicular to the known general south-west to north-east slope present in the gravimetric component of AUSGeoid09 (and more generally the geoid across Australia). While this slope indicates that a small residual geometric effect may be present in this area, the sample is not large enough to identify any anomalies in AUSGeoid09 with any certainty. The small offset between the two models is consistent with the geometric component of AUSGeoid09, generally amounting to about -0.1 m or less in this area (Brown et al., 2011). This suggests that the geometric component of AUSGeoid09 may have overcompensated for the effect of sea surface topography in this case.

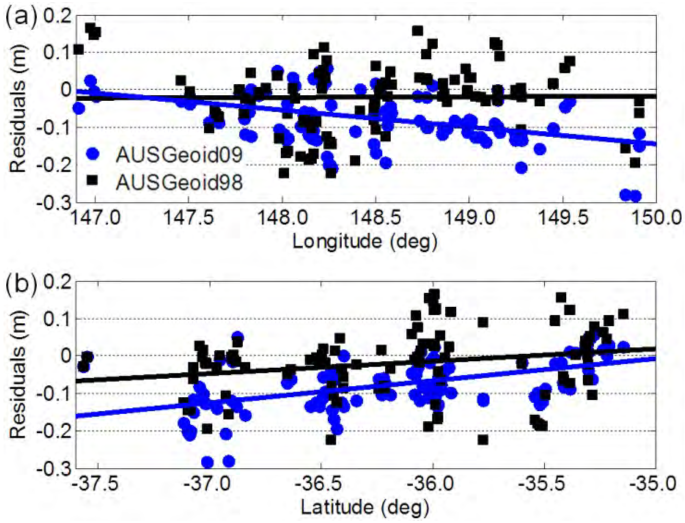

Figure 4: Snowy Mountains verification: Absolute residuals between quasigeoid-derived (AUSGeoid09 and AUSGeoid98) N values and N_{AHD} values for (a) increasing longitudes and (b) decreasing southern latitudes.

6.1.2 Absolute Comparison as a Function of AHD Height

In mountainous regions, it is useful to investigate the performance of the two quasigeoid models as a function of AHD height. While it is recognised that the sample of checkpoints decreases considerably with increasing elevation, this will provide an indication of how well the two models fit AHD in undulating terrain. Following the approach taken by Featherstone and Guo (2001), Figure 5 illustrates the absolute N value residuals for both quasigeoid models as a function of AHD height for the 82 checkpoints located in the Mid Hunter study area. Figure 6 shows the corresponding results for the 104 checkpoints in the Snowy Mountains study area.

It is confirmed that AUSGeoid09 produces a smaller scatter in the residuals and generally provides a better fit (i.e. residuals closer to zero), especially for higher elevations. This is particularly evident in the Mid Hunter study area, clearly showing the improvement obtained when using AUSGeoid09. As previously mentioned, the offset between the two models is consistent with the magnitude of the geometric component of AUSGeoid09. This again demonstrates the benefit of introducing the geometric component, but also indicates that it may have overcompensated for the sea surface topography effect in the Snowy Mountains study area. However, it should be noted that these results are not a true representation of a continuous elevation model because the checkpoints are located across a large area and both datasets contain a small number of checkpoints at the higher elevations.

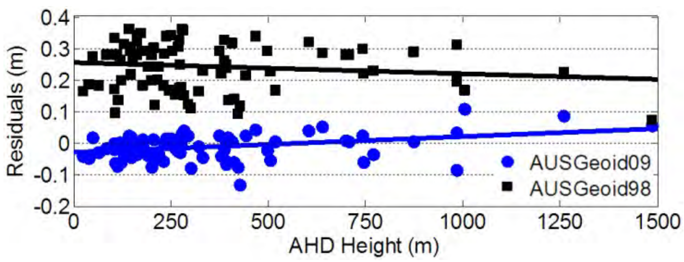

Figure 5: Mid Hunter verification: Absolute residuals between quasigeoid-derived (AUSGeoid09 and AUSGeoid98) N values and N_{AHD} values as a function of AHD height.

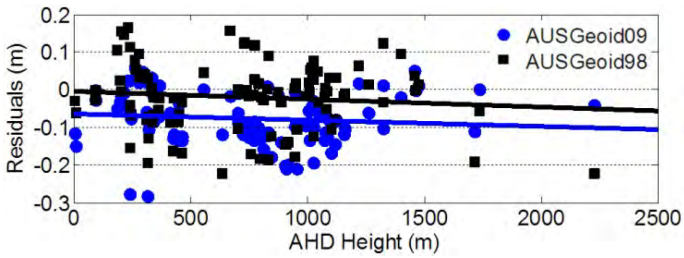

Figure 6: Snowy Mountains verification: Absolute residuals between quasigeoid-derived (AUSGeoid09 and AUSGeoid98) N values and N_{AHD} values as a function of AHD height.

These findings are supported by investigating descriptive statistics of the absolute residuals, calculated for all checkpoints in increments of 200 m in AHD height (values not shown here – for details see Sussanna et al., 2014). In the Mid Hunter study area, both quasigeoid models demonstrate relatively stable and consistent sets of statistics with increasing elevation. AUSGeoid09 shows substantial improvements in the mean, standard deviation, RMS and also the range of residuals. The large positive bias in the mean of the AUSGeoid98 residuals has been successfully accounted for by AUSGeoid09.

In the Snowy Mountains study area, AUSGeoid09 shows improvements over AUSGeoid98 in terms of standard deviation and range, particularly for higher elevations. The AUSGeoid09 statistics are also generally more stable with increasing elevation. However, the RMS only shows improvement for the highest elevations. As mentioned earlier, the mean of the AUSGeoid09 residuals is noticeably biased to the negative.

6.2 Relative Verification

Relative verification is based on GNSS baselines stemming from the constrained least squares adjustment. AUSGeoid09 and AUSGeoid98 performance was investigated by comparing between any two points the difference in GNSS-derived AHD heights (ΔH_{GNSS}) computed using equation (2) with the difference in official AHD heights (ΔH_{AHD}) for these two points. The residuals, computed using equation (3), were then compared in four different tests.

6.2.1 Relative Comparison for all Observed Baselines

In the Mid Hunter network, analysis of the 104 observed baselines shows that AUSGeoid09 and its predecessor AUSGeoid98 perform at the same level as far as recovering AHD heights from GNSS is concerned, with an RMS of about 0.046 m. AUSGeoid09-derived results included no outliers and one outlier was identified when using AUSGeoid98. On average, AUSGeoid98 actually performs slightly better when the average baseline length of 13.1 km is considered (3.7 ppm vs. 4.0 ppm). However, this 0.3 ppm difference is equivalent to less than 4 mm in height over the average baseline length, i.e. less than 10% of the allowable misclose. Both models perform similarly in meeting third-order differential levelling specifications, with about 65% of baselines within specifications. Interestingly, the highest AUSGeoid98 residuals occur at different elevation ranges without any particular pattern, while the highest AUSGeoid09 residuals occur for baselines with height differences between 400 m and 750 m. However, it should be noted that this finding is based on only 6 out of 104 observed baselines and not evident when all possible mark-to-mark combinations are investigated.

In the Snowy Mountains network, analysis of the 66 observed baselines demonstrates that AUSGeoid09 performs moderately better than its predecessor AUSGeoid98. AUSGeoid09-

derived results showed no outliers, while one outlier was identified when using AUSGeoid98. The RMS dropped from 0.064 m to 0.051 m, resulting in an improvement factor of almost 1.3. AUSGeoid09 also performs slightly better when the average baseline length of 7.5 km is considered (7.9 ppm vs. 8.1 ppm) – this 0.2 ppm difference is equivalent to only 1.5 mm in height over the average baseline length (less than 5% of the allowable misclose). AUSGeoid09 produced better results in regards to meeting third-order differential levelling specifications, with about 61% vs. 56% of baselines within the maximum allowable misclose.

6.2.2 Relative Comparison for all Possible Mark-to-Mark Combinations

In the Mid Hunter network, a total of 3,320 possible mark-to-mark vector combinations were analysed. Although AUSGeoid09-derived results identified 15 outliers (none were identified when AUSGeoid98 was applied), it is evident that AUSGeoid09 is far superior to its predecessor. The RMS dropped from 0.105 m to 0.056 m, i.e. an improvement factor of 1.9. Considering the average vector length of 74.9 km, using AUSGeoid09 improved performance by a factor of 1.7, from 1.5 ppm to 0.9 ppm. This 0.6 ppm difference translates into 45 mm in height over the average vector length, thus having a substantial effect (43% of the allowable misclose). Furthermore, 90% of the AUSGeoid09-derived residuals met third-order differential levelling specifications (Figure 7a), while only 61% of AUSGeoid98-derived residuals achieved the same (Figure 7b).

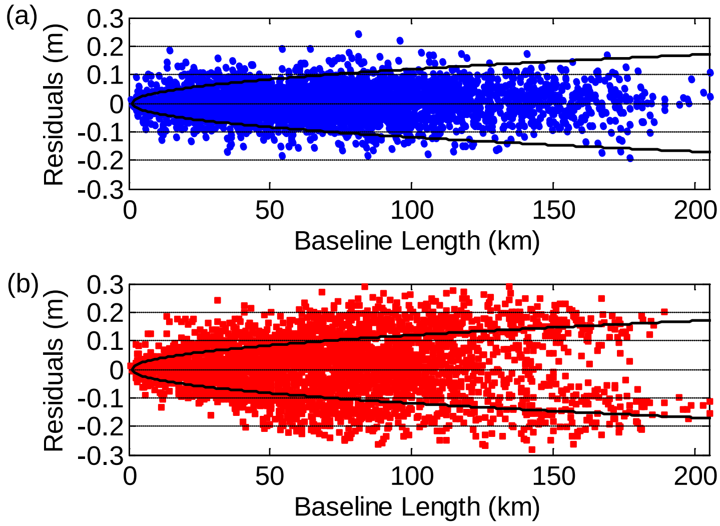

Figure 7: Mid Hunter residuals between (a) AUSGeoid09- and (b) AUSGeoid98-derived and official AHD height differences over 3,320 possible vectors. The unbroken line describes the allowable third-order differential levelling misclose.

In the Snowy Mountains network, a total of 5,356 possible mark-to-mark combinations were analysed. Again, although AUSGeoid09-derived results identified 15 outliers (3 were identified when using AUSGeoid98), it is evident that AUSGeoid09 performs much better than AUSGeoid98. The RMS dropped from 0.127 m to 0.099 m, translating into an improvement factor of 1.3. Considering the average vector length of 113.4 km, AUSGeoid09 improved performance from 1.4 ppm to 1.1 ppm. This 0.3 ppm difference is equivalent to 34 mm in height over the average vector length, i.e. 27% of the allowable misclose. Furthermore, 77% of the AUSGeoid09-derived residuals met third-order differential levelling specifications (Figure 8a), while only 64% of AUSGeoid98-derived residuals fell within these bounds (Figure 8b).

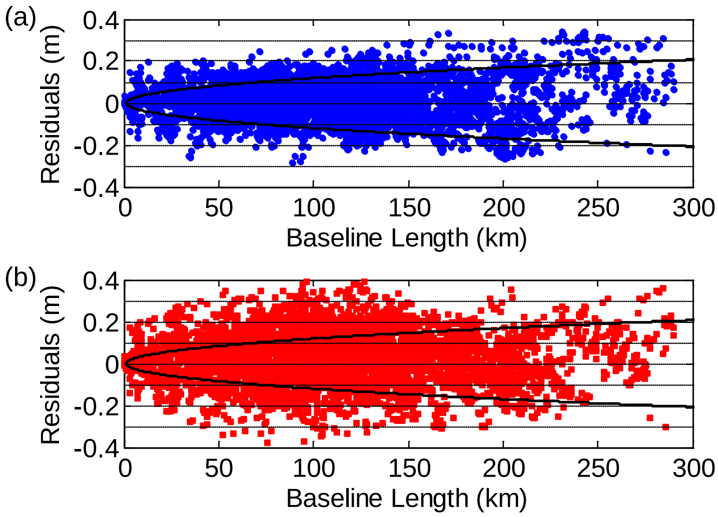

Figure 8: Snowy Mountains residuals between (a) AUSGeoid09- and (b) AUSGeoid98-derived and official AHD height differences over 5,356 possible vectors. The unbroken line describes the allowable third-order differential levelling misclose.

6.2.3 Relative Comparison for all Possible Mark-to-Mark Combinations up to 100 km

The comparison over all possible mark-to-mark combinations (section 6.2.2) includes long vectors that are often well above 100 km in length. Generally speaking, it is unlikely that GNSS users perform network adjustments with baselines of this length, unless they contribute to state-wide or national control networks. Therefore, this third test was conducted to verify the performance of AUSGeoid09 based on the subset of all possible vectors up to 100 km in length, providing a more realistic GNSS network performance evaluation approach from a practical point of view.

In the Mid Hunter network, a total of 2,526 possible vectors with lengths up to 100 km were analysed. Although AUSGeoid09-derived results identified 12 outliers (none were identified when AUSGeoid98 was applied), AUSGeoid09 again shows superior performance to AUSGeoid98. The RMS dropped from 0.095 m to 0.055 m, i.e. an improvement factor of 1.7. Considering the average vector length of 57.4 km, using AUSGeoid09 improved performance from 1.6 ppm to 1.1 ppm (improvement factor of 1.5). This 0.5 ppm difference translates into more than 28 mm in height over the average vector length, thus having a substantial effect (31% of the allowable misclose). Moreover, 87% of the AUSGeoid09-derived residuals met third-order differential levelling specifications, while only 61% of AUSGeoid98-derived residuals achieved the same.

In the Snowy Mountains network, a total of 2,361 possible vectors up to 100 km length were analysed. For both models, a similar number of outliers were identified (7 vs. 6). Again, it is evident that AUSGeoid09 performs much better than AUSGeoid98, with the RMS dropping from 0.122 m to 0.082 m (improvement factor of 1.5). Considering the average vector length of 60.9 km, AUSGeoid09 improved performance from 2.3 ppm to 1.7 ppm. This 0.6 ppm difference is equivalent to more than 36 mm in height over the average vector length, again having a substantial effect (38% of the allowable misclose). In this case, 71% of the AUSGeoid09-derived residuals met third-order differential levelling specifications, compared to only 53% of AUSGeoid98-derived residuals falling within specifications.

6.2.4 Relative Comparison for all Possible Mark-to-Mark Combinations as a Function of AHD Height Difference

Investigating the performance of the two quasigeoid models as a function of AHD height difference provides an indication of how well the two models fit AHD in undulating terrain. Again following the approach taken by Featherstone and Guo (2001), Figure 9 illustrates the residuals for both quasigeoid models as a function of AHD height difference for the 3,320 vectors located in the Mid Hunter study area. Figure 10 shows the corresponding results for the 5,356 baselines in the Snowy Mountains study area.

It is confirmed that AUSGeoid09 produces a smaller scatter or variation in the residuals and generally provides a better fit (i.e. residuals closer to zero). For higher elevation changes, this is particularly evident in the Snowy Mountains study area, clearly showing the improvement obtained when using AUSGeoid09.

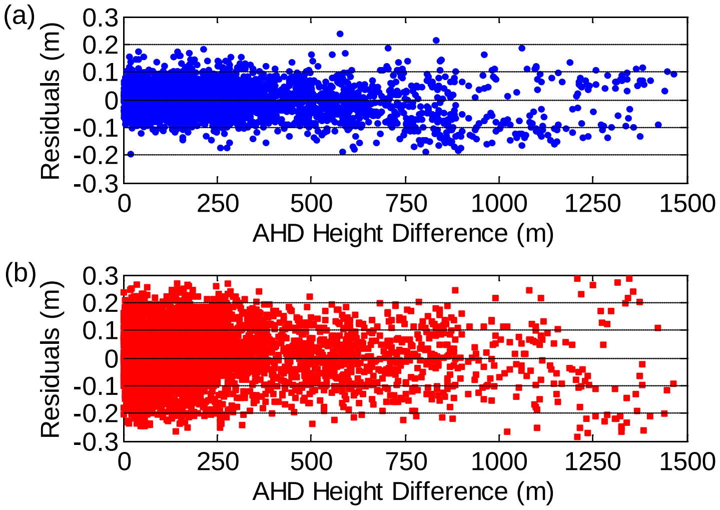

Figure 9: Mid Hunter residuals between (a) AUSGeoid09- and (b) AUSGeoid98-derived and official AHD height differences over 3,320 possible vectors as a function of AHD height difference.

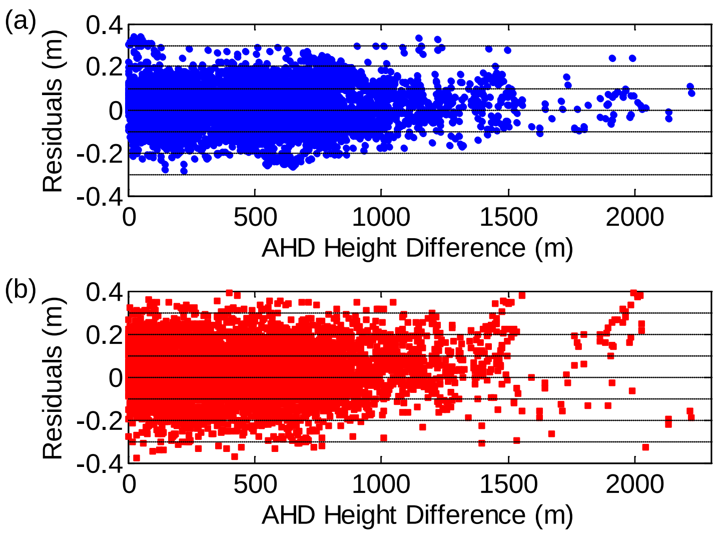

Figure 10: Snowy Mountains residuals between (a) AUSGeoid09- and (b) AUSGeoid98-derived and official AHD height differences over 5,356 possible vectors as a function of AHD height difference.

These findings are supported by investigating descriptive statistics of the residuals between GNSS-derived and official AHD height differences, calculated for all possible vectors in increments of 200 m in AHD height difference (values not shown here – for details see Sussanna et al., 2016). In the Mid Hunter network, both quasigeoid models demonstrate relatively stable and consistent sets of statistics with increasing height differences. RMS values for AUSGeoid09 slowly increase with increasing height difference, and the largest values occur for baselines with height differences of 800 m to 1,200 m. RMS values for AUSGeoid98 are larger but remain consistent until increasing considerably for elevation changes above 1,200 m. AUSGeoid09 shows substantial improvements in the standard deviation, RMS and the range of residuals across all height increments when compared to AUSGeoid98.

In the Snowy Mountains network, both quasigeoid models also present relatively stable and consistent statistics. RMS values for AUSGeoid09 show no evidence of deterioration with increasing elevation change. However, AUSGeoid98-derived results noticeably deteriorate for elevation changes above 1,400 m, indicating a much improved fit to AHD when using AUSGeoid09 for GNSS-based height transfer in heavily undulating terrain. Across all height increments, AUSGeoid09 again shows substantial improvements over AUSGeoid98 in the standard deviation, RMS and the range of residuals.

7 CONCLUDING REMARKS

By examining two extensive datasets located in NSW, this paper has investigated the performance of the AUSGeoid09 quasigeoid model in mountainous regions and compared it to its predecessor AUSGeoid98, from a user's perspective. As expected, AUSGeoid09 has demonstrated increased consistency and accuracy compared to its predecessor, owing to the inclusion of a geometric component, a larger amount of input data and its higher density.

In the absolute verification (i.e. based on absolute N values and AHD heights), this improvement was more evident in the Mid Hunter than in the Snowy Mountains. In the Mid Hunter study area, AUSGeoid09 showed a substantial improvement over AUSGeoid98 in retrieving AHD heights from GNSS-derived ellipsoidal heights, evidenced by the standard deviation dropping from ±0.074 m to ±0.040 m and RMS values improving by a factor of 6. No trend was evident as a function of the horizontal position of the checkpoints, and relatively stable and consistent sets of statistics were obtained for increasing elevations. AUSGeoid09 clearly demonstrated the benefit of its geometric component on GNSS-derived AHD height determination in this area.

In the Snowy Mountains study area, AUSGeoid09 provided moderate improvement over AUSGeoid98, with the standard deviation dropping form ±0.090 m to ±0.070 m. However, it should be noted that the majority of AUSGeoid09 residuals were negative, rather than evenly distributed around a zero mean. This suggests that the geometric component of AUSGeoid09 may have overcompensated for sea surface topography in this case. The dataset also detected a slope in AUSGeoid09 residuals from the north-west to the south-east corner, indicating that a small residual geometric effect may be present in this area. However, the sample size is not sufficiently large to identify any anomalies in AUSGeoid09 with any certainty. It is recognised that the terrain correction (TC) is likely to represent a more prominent error source for the N values in this study area. The accuracy of its computation is dependent on the integrity of the Digital Elevation Model (DEM) in the region, and for the rugged terrain of the

Snowy Mountains this TC element is expected to show a larger degree of uncertainty than in the Mid Hunter.

In this context, it is worth noting that DFSI Spatial Services is currently undertaking the Surface Model Enhancement (SME) Project, which utilises a variety of technology including aerial imagery and Light Detection and Ranging (LiDAR) to create a high-resolution, state-wide Digital Surface Model (Powell, 2017). This is expected to significantly improve the quality of the DEM across the entire State, including mountainous regions.

In the relative verification (i.e. based on N-value and AHD height differences), AUSGeoid09-based processing identified more outliers, but this can be explained by standard deviations generally being substantially smaller than those for the AUSGeoid98-derived results. AUSGeoid09 residuals were better distributed and consistently smaller than AUSGeoid98 residuals, which was particularly evident for large height differences. It was found that AUSGeoid09 generally provides AHD height differences at the ±0.05 m to ±0.09 m level (1 sigma) in the two study areas. Importantly, for practicing surveyors, the use of AUSGeoid09 has substantially increased the percentage of GNSS-derived height differences meeting third-order differential levelling specifications, with up to 90% of AUSGeoid09-derived height difference residuals falling within the maximum allowable misclose. This is a very encouraging result, considering the well-known difficulties of spirit levelling in mountainous terrain and the increasing popularity of GNSS-based height transfer in practice.

It is important to note that the results presented do not provide a general verification of AUSGeoid09 in other mountainous regions. It is acknowledged that sources of error exist within extensive datasets, even after careful investigation. Furthermore, GNSS and AHD height data have their own error budgets, and the sparseness of gravity data and any uncorrected biases in the spirit-levelling data in mountainous terrain may have contributed to some of the trends shown. However, considering that the main use of AUSGeoid09 is to compute AHD heights from GNSS-derived ellipsoidal heights, the data and method employed in this paper represent the most practical means of quasigeoid verification currently available.

The positive results of AUSGeoid09 performance in mountainous regions are encouraging, particularly in light of GNSS technology and CORS networks being increasingly used to provide vertical control. It can be expected that AUSGeoid2020, in conjunction with GDA2020, will provide further improvements for GNSS users, owing to improved modelling (including terrain modelling) and larger input datasets. To this end, DFSI Spatial Services continues to collect extended GNSS datasets on levelled marks across NSW (e.g. Gowans et al., 2015; Gowans, 2017; Janssen, 2017). The methodology presented in this paper will be very beneficial in regards to future testing of AUSGeoid2020. Eventually, however, the introduction of a new national vertical datum for Australia will be necessary in order to achieve higher consistency and generate a vertical reference surface that is more closely aligned to the geoid.

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Impact of the Proposed Surveying and Spatial Information Regulation 2017

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ABSTRACT

In New South Wales, practising surveyors are subject to the Surveying and Spatial Information Act 2002 and the Surveying and Spatial Information Regulation 2012. Pursuant to Section 10 of the Subordinate Legislation Act 1989, the Surveying and Spatial Information Regulation 2012 is due to be repealed on 1 September 2017 and replaced by a new Surveying and Spatial Information Regulation 2017. The objectives of the Surveying and Spatial Information Regulation are to ensure the competency of surveyors, maintain the integrity of the cadastre for New South Wales and ensure measurement and marking standards are delivered from modern surveying and communication technologies. Key outcomes of the proposed Surveying and Spatial Information Regulation 2017 are greater enablement of digital government and greater integration of positioning. This paper outlines a number of key reforms introduced in the proposed Surveying and Spatial Information Regulation 2017. These include specifications for positioning outcomes, greater integration with the Map Grid of Australia and the Australian Height Datum and centralising of information workflow to support digital government, digital business and e-Plan automation.

KEYWORDS: *Regulation, positioning, integration, digital government, e-Plan.*

1 INTRODUCTION

Under the *Subordinate Legislation Act 1989* (NSW Legislation, 2017c), all statutory rules (i.e. regulations) must be remade every 5 years to ensure they remain relevant and current to government, community and industry needs. The *Surveying and Spatial Information Regulation 2012* (NSW Legislation, 2017b) is due to be repealed on 1 September 2017. Consultation with the surveying and spatial information industry during 2016 and 2017 ensure that emerging issues and needs are addressed in the remake of the Regulation.

The Regulation is made under the *Surveying and Spatial Information Act 2002* (NSW Legislation, 2017a). The Act incorporates all aspects of the Regulation and oversight of land and mining surveying in NSW. The major objectives of the Act are to:

- Ensure the accuracy and integrity of the State cadastre that "enables people to readily and confidently identify the location and extent of all rights, restrictions and responsibilities

related to land and real property" (ICSM, 2015).

- Maintain and develop the state control survey, which provides a reliable and accurate spatial referencing system underpinning surveying, land information and mapping systems in NSW.
- Provide a framework for registration and coordination of surveys by public authorities.

In order to achieve these objectives, the Act requires that surveyors must be registered and must comply with minimum standards of education and competency. The Act establishes the Board of Surveying and Spatial Information (BOSSI, 2017) to oversee the registration of surveyors, set professional education requirements and conduct disciplinary investigations to ensure consistency and quality in the delivery of surveying services.

The key reforms of the proposed *Surveying and Spatial Information Regulation 2017* are:

- Greater enablement and integration of positioning through greater integration with the Map Grid of Australia (MGA) and the Australian Height Datum (AHD).
- Enablement of digital government, digital business and E-Plan automation by centralising information on Deposited Plans.

2 GENERAL PROCESS

During 2016 and 2017, several workshops and presentations, including consultation with industry representatives, have been conducted to determine the issues and principles within the *Surveying and Spatial Information Regulation 2012* that needed reform. From these workshops and presentations, a detailed working brief has been forwarded to the Parliamentary Counsel's Office with a request to draft a new consultation draft Regulation. In addition to the draft Regulation, a Regulatory Impact Statement (RIS) will also be prepared. The objective of the RIS is to outline who is making the new Regulation, why it is being made and to consider all options for the proposed changes to ensure the best outcome is to be achieved. The RIS will weigh up the costs and benefits of the proposed Regulation and also consider alternative options for achieving the required outcomes.

The RIS, together with the consultation draft Regulation, will be advertised in the media, sent to all relevant surveying and titling industry groups (such as the Institution of Surveyors NSW, Law Society of NSW, government agencies, etc.) and be available for download by the public of NSW. The intent of the advertising is to ensure that all surveyors and participants in the surveying and spatial information industry are aware of the draft changes and to invite submissions on the draft Regulation. Both documents are anticipated to be available for public and industry consultation during March 2017. The submissions received may, after appropriate consideration, result in amendments to the draft Regulation being made prior to approval of the final Regulation by the Minister and Surveyor-General of NSW.

3 SUMMARY OF PROPOSED CHANGES AND ANTICIPATED IMPACT

A summary of the proposed major changes from the *Surveying and Spatial Information Regulation 2012* to the *Surveying and Spatial Information Regulation 2017* are outlined in Table 1. The sequence or numbering of the changes is based upon the clause numbering in the draft 2017 Regulation.

Table 1: Summary of major changes from the 2012 Regulation to the 2017 Regulation.

2017 Clause Number and Title	2017 Regulation	Reason for Change
4 Mining surveys	References to legislation updated to correspond with current legislation	References to legislation must remain current so Clause 4 retains validity.
5 Definitions The definitions shown on the right were added or changed.	*Accurate AHD value* – defined to be an AHD value in SCIMS equal to or better than Class "B" or Class "LD".	*Accurate AHD value* – refined for clarity; the addition of the "in SCIMS" qualifier has been added so that there is only one single source of truth for accurate height values within NSW.
	Established survey mark – definition changed to Class "D".	*Established survey mark* – established survey mark changed to Class D to enable more Deposited Plans to be placed on MGA while retaining the integrity required for easy integration within the Foundation Spatial Data Framework; it is an enabler for digital government and network propagation.
	Mean low water mark – possible addition of definition or reference.	*Mean low water mark* – as a result of several registered Deposited Plans, there may be a requirement for "mean low water mark" to be added or referenced as some registered Deposited Plans have referenced "mean low water mark". See: DP1043662, DP1128433 & DP1215295.
	Positional uncertainty – definition added from *Standards and Practices for Control Surveys (SP1) (Version 1.7)* published in September 2007 by the Intergovernmental Committee on Surveying and Mapping (ICSM, 2007).	*Positional uncertainty* – the definition has been included so to express part of the positioning outcomes required as part of the reforms.
	Road – definition expanded.	*Road* – accessways for Community Scheme have been included so that all appropriate marking requirements are met for land effectively used as a road.
	Spline – definition added as part of the change to Clause 65 regarding the method of showing natural feature boundaries. The minimum spline classification that the definition requires is a commonly used form of a spline, that being a piecewise polynomial known as an interpolating cubic spline. Therefore, to comply with the definition, a piecewise polynomial of degree three or higher is required.	*Spline* – advice from the spatial information industry indicates that a spline is considered the geometric entity best suited for the mathematical and graphical representation of a natural feature boundary.
	Urban survey – definition updated to correspond with current legislation.	*Urban survey* – in order to remain current, the definition has been updated to reflect the current planning zones under the Environmental Planning and Assessment Act 1979 as found in the Standard Instrument – Principal Local Environment Plan.

2017 Clause Number and Title	2017 Regulation	Reason for Change
12 Datum line	Clause 12 has been restructured and changes made regarding the adoption of datum lines. Subclause 12(2) has been changed to reference four cases regarding the adoption of a datum line for urban surveys: • Case 1: If the land surveyed is less than 300 m from two established marks, then the grid bearing derived from the SCIMS coordinates of those marks must be used as orientation. **No change from the 2012 regulation.** • Case 2: If Case 1 does not apply and **if the surveyor chooses**, adopt orientation from the SCIMS coordinates of two established marks if the marks are less than 1,500 m from the land surveyed. • Case 3: If Cases 1 & 2 do not apply and the surveyor has used an approved GNSS method, then: orientation must be derived from the MGA coordinates, as determined by an approved GNSS method, of two unestablished survey marks less than 300 m from the land surveyed. • Case 4: If Cases 1, 2 & 3 do not apply, then orientation must be adopted from a plan on public record. Subclause 12(3) now references two cases regarding the adoption of a datum line for rural surveys: • Case 1: If the land surveyed is less than 1,000 m from two established marks, then the grid bearing derived from the SCIMS coordinates of those marks must be used as orientation. **No change from the 2012 regulation.** • Case 2: If Case 1 does not apply, the surveyor may either (i) If the land surveyed is less than 5,000 m from two established marks, derive orientation from the grid bearing derived from the SCIMS coordinates. or (ii) Derive orientation from the MGA coordinates, as determined by an approved GNSS method, of two unestablished survey marks less than 1,000 m from the land surveyed.	To place as many plans as possible on an MGA orientation. The placing of as many surveys as possible on an MGA orientation is an enabler of digital government and integration of position into the Foundation Spatial Data Framework. It is reasonable to expect a survey utilising an approved GNSS method to be able to place the survey onto an MGA orientation. The initial desired outcome was for all plans to be placed on an MGA orientation. This proposal was problematic in urban areas, where there exist significant gaps in the “established” network and GNSS usage is less prevalent.

2017 Clause Number and Title	2017 Regulation	Reason for Change
	This effectively means that all rural surveys will be on an MGA orientation whether via established marks or approved GNSS methods. The tolerance specified in the 2012 subclause 12(6) for the verifying line has been loosened to 40 mm + 175 ppm. The 2012 Cl. 12(7)(c) placed in Clause 12(8) and altered to encompass all approved GNSS methods referred to in all clauses.	 A looser tolerance is required to accommodate the expansion of the *established survey mark* definition to Class D. A regulation preventing the use of unchecked GNSS baselines should apply to all approved GNSS methods used.
13 Bench marks	References to "external" bench marks have been removed. Clause 13(6) regarding the determination of the position of each bench mark has been removed.	The stipulation of "external" bench marks is not considered necessary for the vast majority of surveys and removal improves interpretation of the Clause. This stipulation has been placed in Clause 71.
23 Accuracy of angular measurement	Clauses 23 & 24 have been combined. An accuracy of included angles has been inserted as subclause 23(7): If the bearings of two lines shown on the survey plan have a common vertex, the accuracy of the included angle must be within the tolerance of: $206265\left(\frac{0.01+\left(\frac{d}{20000}\right)}{d}\right)$ seconds of arc, where *d* is the length in metres of the shortest line.	Combining the same subject matter for clarity. Individual bearings shown on the plan should be accurate to a minimum standard in the same way that individual lengths need to be. The angular misclose and parcel misclose regulations do NOT regulate the accuracy of individual bearings or individual angles shown on a plan.
25 Accuracy of relative position	An accuracy of relative position has been inserted as Clause 25: When conducting a survey, a surveyor must ensure that the accuracy of the relative positions between any two surveyed points is within the tolerance of: $\sqrt{2\left(0.01+\frac{d}{20000}\right)^2}$ metres where *d* is the distance in metres between the points.	Relative positions of surveyed points on a plan should be accurate to a minimum standard just as individual lengths need to be and individual bearings and angles should be. The community expects database tools to be available and be accurate. Integration of surveys into databases requires integrity of shape as well as position – the accuracy of relative position gives the integrity of the shape of the survey.

2017 Clause Number and Title	2017 Regulation	Reason for Change
28 Boundary marks	Inserted as subclause 28(3)(b), regarding inaccessible corners: If the corner that cannot be marked is within a structure or is otherwise made inaccessible by a structure, the corner does not require a reference mark to be placed and must be shown by the appropriate symbol depicted in Schedule 5 (solid circle).	The requirement to place a reference mark at a corner that is within or made inaccessible by a structure (e.g. a corner within a party wall) is considered unreasonable.
29 Reference marks for urban surveys	Clarified so that “extremity” can also include the junction and intersection of roads. Intervening side boundaries subclause clarified.	The junction or intersection of a road in an urban survey was intended to be included as an extremity of the land surveyed; the change explicitly states this and removes doubt.
35 Surveyor to note nature and position of survey marks etc.	All references to how coordinates and heights should be shown on the plan have been moved to Clauses 69, 70 & 71 – “Division 7 – Survey plans”.	This clause is within “Division 4 Use of survey marks and monuments”. All requirements regarding what needs to be shown on the survey plan should be within “Division 7 – Survey plans”. Coordinate and height schedules have been consolidated into one area of the Regulation for ease of interpretation and clarity of purpose.
42 Connection to permanent survey marks	Inserted as a new subclause 42(4): Permanent survey marks used only to comply with Clause 13(2) (verification of AHD for bench marks) do not need to comply with: • Subclause 42(3) (connections to the land proved by closed survey). • Subclause 70(2)(b)(ii) (MGA coordinate shown to Class “D”).	If a permanent survey mark is used only for compliance with 13(2), then it is unreasonable to expect the surveyor to connect the mark to the land surveyed by closed horizontal connection, or to coordinate the mark to Class D standard for the purposes of the coordinate schedule.
60 Method of recording datum line	All references to how coordinates and heights should be shown on the plan have been moved to Clauses 69, 70 & 71. Subclauses added to stipulate that: • The survey plan must state from what the orientation of the survey has been derived (from Cl. 12). • The datum line and any verifying line must (if practicable) be related to the survey by closed connection. • Comparisons with SCIMS or the previous plan must be shown for the datum line.	Consolidation of all matters regarding showing the datum line on the survey plan. Explicit stipulations of datum line comparisons for all cases are not present in the current regulation; comparisons are needed as an affirmation of the datum line validity.
65 Method of showing natural feature boundaries	A survey plan that shows a natural feature boundary must indicate the boundary by a spline curve, not an “irregular line”. A survey plan that shows a natural feature boundary must show the connection between terminals of the natural feature for each lot.	Advice from the spatial information industry indicates that a spline is considered the geometric entity best suited for the mathematical and graphical representation of a natural feature boundary. See spline definition above. To facilitate ease of checking of natural feature boundary closes, especially where a lot boundary intersects the natural feature boundary. Clearly defines the point at which the side boundary of a lot cuts the natural feature boundary.

2017 Clause Number and Title	2017 Regulation	Reason for Change
66 Survey plan to show GNSS validation	The requirement to show GNSS derived lines in the previous (2012) Regulation has been removed. The Clause has been changed to stipulate that if an approved GNSS technique is used in the survey, then details of the GNSS validation must be shown on the plan of survey. A further requirement is for surveys adopting an MGA orientation from MGA coordinates obtained by an approved GNSS method. In this case, the GNSS validation required above must be performed and shown for the datum line of orientation.	All GNSS are operated by international parties. Most GNSS augmentation systems (e.g. CORSnet-NSW, GPSnet) are operated by Government or commercial third parties. These are NOT under the surveyor's direct control. As such, any GNSS equipment and methods used must be confirmed against an independent external source of known accuracy. As, in this case, the datum line is based only on coordinates obtained by an approved GNSS method, those coordinates should be independently checked; the best method of achieving this check is via the GNSS validation process.
69 Survey plan to show height differences	A new requirement to show height differences in a schedule on the survey plan for new permanent marks placed; the permanent survey marks to which this clause applies are those that require AHD to be determined under Clause 13 or Clause 43(2).	The height datum needs to be propagated for infrastructure management and development; survey control is essential public infrastructure, in just the same way as sewer, stormwater drainage, electricity, telephone and data services. Height is an important component of survey control. The body best placed to propagate the height datum is the surveying industry; the industry has the requisite professional skills and local knowledge to effectively propagate height in their local area. The plan of survey is being used as the delivery mechanism for height from the surveying industry for several reasons: • The previous information delivery mechanism was through Locality Sketch Plans; Spatial Services have received less than 65% of Locality Sketch Plans for marks placed in 2016, so this delivery mechanism has demonstrably failed. • The information is in one location, facilitating e-Plan validation and Spatial Services' collation and ingestion of the information. • In specifying height differences and height values to be shown, the survey plan becomes self-describing and self-checking.

2017 Clause Number and Title	2017 Regulation	Reason for Change
70 Survey plan to show coordinate schedule	The requirements for showing of the coordinate schedule have been consolidated into one clause. If a survey adopts an MGA orientation, then coordinates need to be shown to Class D or better for unestablished marks.	Consolidation of coordinate schedule requirements into one clause facilitates clarity and ease of understanding. If a mark has been located by the surveyor in a closed survey and the survey has connected to MGA, then it is not unreasonable to ask the surveyor to provide Class D MGA coordinates for the unestablished marks instead of hand-held GNSS coordinates. This enables better initial positioning of the mark.
71 Survey plan to show height schedule	The requirements of clause 35 & 62 in the 2012 Regulation regarding height schedules have been consolidated into the new clause 71. A new classification for marks with accurate AHD values (those in SCIMS) of "height datum validation" has been added; the classifications can be either "SCIMS adopted" or "from SCIMS – datum validation" where the classification "SCIMS adopted" can only be used once.	Consolidation of height schedule requirements into one clause facilitates clarity and ease of understanding. The method used to derive the AHD value is not needed as it will be a derivative of the method/s shown in the height difference table. Of more importance is the height datum validation – the SCIMS AHD value adopted to derive the AHD values for marks not having an "accurate AHD value" and the marks used to validate the "accurate AHD value" of the single mark adopted. The plan of survey is being used as the delivery mechanism for height from the surveying industry for the reasons listed above.
81 Provision of further information and supporting evidence	An amendment to subclause 81(e): an applicant to the Board for registration as a surveyor must furnish a recent photograph of the applicant's face that meets the specification required for an Australian passport photograph.	Standardization of the photograph specification.
90 Applications to remove survey marks under section 24 of the Act	Subclause 90(2) has been altered to include bench marks.	Bench marks are important survey infrastructure and should be protected.

2017 Clause Number and Title	2017 Regulation	Reason for Change
91 Exemption by the Surveyor-General	New subclause 91(3): If a survey plan to which an exemption applies is lodged with a public authority, then that public authority must be furnished with a copy of the exemption. New subclause 91(4): Any survey subject to an exemption must comply with all conditions contained within the exemption.	Exemptions are often issued subject to one or more conditions. When the plan is lodged with a public authority, the plan currently only contains a reference noting which clause the exemption applies to, not the specifics of the exemption (it may only apply to a part of the plan, not the whole) nor the conditions, if any, of the exemption. This means when examining the plan, the examiner has no way of knowing to what part of the survey the exemption applies, nor whether any of the conditions contained within the exemption have been complied with. Requiring the surveyor to lodge a copy of the exemption with the plan removes this issue.
Schedule 1 Bench marks	"Bench Mark token" added.	Augmentation of bench mark types with a minor modification to an existing, easy to use and readily identifiable mark type (the existing mark type being a "Boundary Mark token").
Schedule 2 Boundary marks	Uncapped fixed steel fence post added as an approved mark – no reference mark necessary. Punch Mark added. When referencing a specific point on a structure to a corner that abuts a road, an additional reference mark must be placed within the road corridor.	It is very difficult to mark uncapped steel posts at a cadastral corner. Augment available options for marking. Removes the issue where the corner of a building used to reference a corner abutting a road is inaccessible.
Schedule 3 Reference marks	"Reference Mark token" added. Non-corrodible nail (concrete) (20 mm long) has been deleted.	Augmentation of bench mark types with a minor modification to an existing, easy to use and readily identifiable mark type (the existing mark type being a "Boundary Mark token"). Stability of the mark has proven to be not satisfactory.
Schedule 5 Conventional signs and symbols	Schedule updated.	To retain currency and consistency with standard usages and reforms.

Clause 55 of the *Surveying and Spatial Information Regulation 2012* – Surveyor to record astronomical observations – has been removed, as have all references to astronomical observations as very few surveyors utilise astronomical observations in daily practice. The Regulation must reflect the changing practices of the surveying profession to remain relevant.

The clauses within the proposed *Surveying and Spatial Information Regulation 2017* most affected by the proposed changes are:

- Clause 12 – Datum line.
- Clause 23 – Accuracy of angular measurement.

- Clause 25 – Accuracy of relative position.
- Clauses 69, 70 & 71 – Coordinate and height schedules.

The major impacts of the changes in the proposed *Surveying and Spatial Information Regulation 2017* will probably be:

- More survey plans on an MGA orientation.
- All rural surveys to be on an MGA orientation.
- Alteration of the delivery mechanism for height by centralising of all height information on the survey plan.
- Enablement of digital government, digital business and e-Plan automation by centralising information on Deposited Plans.

4 CONCLUDING REMARKS

The *Surveying and Spatial Information Regulation 2012* is due to cease operation on 31 August 2017 and the *Surveying and Spatial Information Regulation 2017* is proposed to commence on 1 September 2017. It is proposed that any survey completed after 1 September 2017 must satisfy the requirements of the *Surveying and Spatial Information Regulation 2017*. This paper has outlined the changes that are proposed in the *Surveying and Spatial Information Regulation 2017* and the anticipated impact these changes will have on the surveying profession.

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Native Title, Compensation and the Public Authority Land Manager

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ABSTRACT

Two recent native title cases provide a timely reminder to surveyors and public authority land managers about the impact of native title on public land. On 24 August 2016, Justice Mansfield of the Federal Court of Australia, in the case of Griffiths v. Northern Territory of Australia, ordered the Northern Territory Government to pay compensation to the native title holders of Timber Creek for the impact of land grants and public works on their non-exclusive native title rights and interests. This order, which is the first of its kind in Australia to consider the valuation methodology, calculated total compensation at $3,300,661. In general, it is the Commonwealth, State and Territory Governments which are liable to pay compensation for the extinguishment or impairment of native title, depending on which Government has undertaken the act identified as causing the extinguishment or impairment. However, States and Territories can 'pass on' this liability to third parties in certain circumstances by either legislation or under contracts. This can have consequences for local government authorities and other statutory entities who compulsorily acquire native title as well as project proponent corporations and infrastructure providers. On 22 January 2016, in the Federal Court case of Doyle on behalf of the Iman People #2 v. State of Queensland, Justice Reeves considered the expert evidence of an experienced surveyor describing relevant surveying and mapping practices to assist in determining whether native title had been extinguished by the dedication of certain land as a public road. Surveyors, including those with public land management responsibilities, need to be aware of these decisions and any potential consequences they may have for their practice.

KEYWORDS: *Native title, compensation, public land managers*.

1 INTRODUCTION

It is over 20 years since the *Native Title Act* 1993 (Cth) was introduced. While native title has more recently been determined in parts of far western NSW and the far and mid north coast of NSW, the next wave of developments in native title law, namely native title compensation, is only now being examined by the courts. The recent decision in *Griffiths v. Northern Territory (No. 3)* [2016] FCA 900 (Griffiths #3) provides an insight into the methodologies used by one experienced native title judge, Justice Mansfield, to value native title. While the *Native Title Act* 1993 (Cth) attributes compensation liability to the Commonwealth, State and Territory Governments for the extinguishment or impairment of native title, depending on which Government has undertaken the act identified as causing the extinguishment or impairment, it

is possible for this liability to be 'passed on' to third parties in certain circumstances by either legislation or under contracts. This can have consequences for local government authorities and other statutory entities who compulsorily acquire native title as well as project proponent corporations and infrastructure providers. With an increased focus on native title compensation, governments and other entities will be keen to limit their exposure to pay compensation. For this reason, governments will seek to evidence extinguishing acts that occurred prior to the commencement of the *Racial Discrimination Act* 1975 (Cth) on 31 October 1975, being the date from which native title compensation arises. On 22 January 2016, in the Federal Court case of *Doyle on behalf of the Iman People #2 v. State of Queensland*, Justice Reeves was assisted by the expert evidence of an experienced surveyor regarding survey and mapping practices in deciding that native title had been extinguished by the dedication of land as a public road prior to 1975. Surveyors, including those with public land management responsibilities, need to be aware of these decisions and any potential consequences they may have for their practice.

2 A QUICK HISTORY REFRESHER

2.1 Terra Nullius

According to the international law of Europe in the late 18th century, there were only three ways that Britain could take possession of another country:

- If the country was uninhabited, Britain could claim and settle that country. In this case, it could claim ownership of the land.
- If the country was already inhabited, Britain could ask for permission from the indigenous people to use some of their land. In this case, Britain could purchase land for its own use but it could not steal the land of the indigenous people.
- If the country was inhabited, Britain could take over the country by invasion and conquest – in other words, defeat that country in war. However, even after winning a war, Britain would have to respect the rights of indigenous people (Butler et al., 1995).

However, international law was given scant regard in times of colonial expansion and in that regard Britain was no better than the Spanish or the Portuguese in the Americas. From the time of Cook's arrival, Britain acted as if Australia was uninhabited (Figure 1).

Figure 1: The landing of Captain Cook at Botany Bay in 1770 (E. Phillips Fox, ca. 1960).

Over the next 200 years, settlement of New South Wales continued. The various iterations of the Surveyor General's Department and the Department of Lands created portions of land that were alienated from the Crown (sold or leased), or they were reserved for a public purpose (Figure 2). To date 42% of the State is still Crown Land and, even though the bulk of this land is in the Western Division of the State, around 8% of the Eastern and Central Divisions are Crown land.

Figure 2: Early view of a surveyors' camp (Pickering Brook Heritage Group, 2016).

2.2 Aboriginal Rights

2.2.1 The Racial Discrimination Act 1975

The *Racial Discrimination Act* 1975 (Cth) aimed to ensure that people of all backgrounds are treated equally and have the same opportunities. The Act also makes discrimination against people on the basis of their race, colour, descent or national or ethnic origin unlawful. This is significant as it effects a date from which compensation may be payable under the *Native Title Act* 1993 (Cth).

The *Racial Discrimination Act* 1975 makes racial discrimination unlawful in Australia and overrides inconsistent State and Territory legislation, making the State or Territory law ineffective to the extent of the inconsistency. The power of the Commonwealth Parliament to pass this overriding law arises under the "external affairs" power contained in the Australian Constitution (Morrison, 2016). The power arose from the International Convention on the Elimination of all Forms of Racial Discrimination.

2.2.2 The Aboriginal Land Rights Act 1983

The *Aboriginal Land Rights Act* 1983*,* introduced by the Wran Government in New South Wales, commenced on 10 June 1983. In establishing the *Aboriginal Land Rights Act* 1983, the NSW Government acknowledged:

- Land in the State of New South Wales was traditionally owned and occupied by Aborigines.
- Land is of spiritual, social, cultural and economic importance to Aborigines.
- It is fitting to acknowledge the importance that land has for Aborigines and the need of Aborigines for land.
- It is accepted that, as a result of past government decisions, the amount of land set aside for Aborigines has been progressively reduced without compensation (Figure 3).

Figure 3: On 26 August 1975, Prime Minister Gough Whitlam handed a leasehold title to land at Daguragu (Wattle Creek) to Vincent Lingiari, representative of the Gurindji people. It was a turning point for Aboriginal land rights in Australia (photo courtesy of Mervyn Bishop).

The *Aboriginal Land Rights Act* set up the NSW Aboriginal Land Council structure. In NSW, land rights are granted in the form of freehold estate where the Minister administering the *Crown Lands Act* 1989 finds that the land is claimable land for the purposes of the *Aboriginal Land Rights Act.* The *Aboriginal Land Rights Act* establishes a process and conditions for Aboriginal Land Councils to claim land in NSW (Figure 4).

Figure 4: The New South Wales Aboriginal Land Council makes the first land claim pursuant to the new Aboriginal Land Rights Act 1983 at Goanna Headland (NSW North Coast). It is later granted in 1985.

Essentially, claimable Crown lands are lands vested in Her Majesty that:

- Are able to be lawfully sold or leased, or are reserved or dedicated for any purpose, under the *Crown Lands Act*.
- Are not lawfully used or occupied.
- Do not comprise lands which, in the opinion of the Minister, are needed or are likely to be needed as residential lands.
- Are not needed, nor likely to be needed, for an essential public purpose.
- Do not comprise lands that are the subject of an application for a determination of native title that has been registered in accordance with the *Native Title Act*.
- Do not comprise lands that are the subject of an approved determination of native title, other than an approved determination that no native title exists in the lands.

The Minister does not have any discretion when determining Aboriginal land claims and land is claimable under conditions that have been modified by the Courts since the commencement of the *Aboriginal Land Rights Act*. If a land claim is lodged over Crown land that meets the conditions above, the Minister is required to grant the claim.

On 1 July 2015 the *Aboriginal Land Rights Act* was amended to allow the Minister and Aboriginal Land Council(s) to enter into Aboriginal Land Agreements with a view to, among other things, allow parcels under claim to be 'determined' by negotiation rather than on a parcel by parcel basis. The mechanisms for these agreements are currently being developed.

2.2.3 Mabo

On 20 May 1982, Eddie Mabo (Figure 5) and two other Meriam people from the Murray Islands in the Torres Strait lodged a statement of claim in the High Court of Australia. They claim 'native title' rights to the Murray Islands.

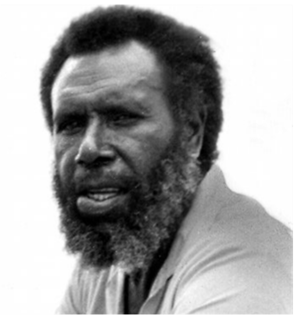

Figure 5: Eddie Mabo.

On 3 June 1992, the High Court recognised that native title is part of Australian land law. The historic decision overturns the doctrine that Australia was terra nullius – a land belonging to no-one. The High Court recognises that the Meriam people were "entitled as against the whole world, to possession, occupation, use and enjoyment of the lands of the Murray Islands."

2.2.4 The Native Title Act 1993

On 19 December 1993, the Federal Parliament enacted the *Native Title Act* 1993. It addresses the consequences of recognising native title for past actions of governments and sets up rules for future dealings in native title land and waters. The legislation followed lengthy debate and negotiations between Indigenous stakeholders, governments, pastoralists and the mining industry. In the second reading speech, former Prime Minister Paul Keating said "the *Native Title Act* does not act to lock land away. It gives certainty to both native title holders and the community about the use of land into the future."

On 23 December 1996, in the *Wik People v. Queensland* case, the High Court decided that native title is not necessarily extinguished by the grant of a pastoral lease and that native title can co-exist with other interests in land.

On 7 April 1997, for the first time, native title is determined on the Australian mainland under the *Native Title Act*, in "The Dunghutti People Consent Determination". In 2010, the Dunghutti Elders Council received $6.1 million as compensation for 12.4 ha of land at Crescent Head that has been used for residential development.

3 NATIVE TITLE

3.1 What is Native Title?

Native title is a communal bundle of rights, and not an individual proprietary right. It depends for its existence on the continuing acknowledgement and observance of the relevant traditions, customs and practices of the community (*Griffiths #3* at [219]). Native title rights may exist over all land and water in Australia unless there has been an act of prior 'exclusive possession'.

Matters relating to native title fall within the Commonwealth jurisdiction of the *Native Title Act 1993* (Cth). All States and Territories, including NSW, have enacted legislation to adopt the Commonwealth scheme. Native title claims are applications filed in and administered by the Federal Court of Australia. The claims are referred to the National Native Title Tribunal to allow public notification of the claim to occur.

Native title deals with the legal recognition of the traditional native title rights and interests that Aboriginal people have in land and water, where Aboriginal people have continued to exercise their rights and interests in accordance with traditional law and custom (NSW Trade & Investment, Crown Lands, 2014).

3.1.1 Native Title Rights

Native title rights will vary depending on an individual group's traditional laws and customs but generally will include the right to:

- Access and occupy the land.
- Camp on the land.
- Live on certain land.
- Hunt, fish, gather and use resources from the land.
- Gather bush medicine.
- Perform traditional ceremonies on the land.
- Possess the land for particular traditional customs.
- Speak for country.

3.1.2 Extinguishment of Native Title

Generally, native title rights will continue to exist unless they have been 'extinguished' prior to 23 December 1996, or cease to exist, as a result of the following:

- The native title claim group has failed to continue to observe their traditional laws and customs, or where they fail to demonstrate continued observance.
- An exclusive possession property interest by the Crown that is wholly or partially inconsistent with the native title rights. This type of interest includes:
  - A grant of freehold estate.
  - A grant of 'Scheduled Interest' as listed in the *Native Title Act*.
  - A commercial lease that is neither an agricultural lease nor a pastoral lease.
  - A residential lease.
  - A community purpose's lease.
  - The construction of a public work.
  - A dedication of a public road.
  - An exclusive agricultural lease or an exclusive pastoral lease.

- o Any lease (other than a mining lease) that confers a right of exclusive possession over particular land or waters.

3.2 Land Where Native Title Applies

Thus, native title may continue to exist in Crown land, including vacant and unallocated Crown land, Crown reserves, Crown land under a permissive occupancy or licence, and land under non-exclusive lease. However, it is not limited to Crown land and can continue to be recognised in:

- National Parks, Marine Parks and State Forests.
- Water bodies such as rivers and lakes and areas below Mean High Water Mark, including parts of the territorial sea, the beds and banks of the waterways.

Increasingly, however, the State Government has transferred Crown land subject to native title rights and interests to other entities. In 1994 with the introduction of the *Native Title (New South Wales)* 1994, the *Aboriginal Land Rights Act* 1983 was amended allowing land claims to be granted to Aboriginal Land Councils subject to native title rights and interests. In 2006, the *Government Property NSW Act* allowed the transfer of Crown land with native title intact to that agency. Various single site acts establishing management entities such as the *Parramatta Park Trust Act 2001* were created maintaining existing native title rights and interests.

Most recently, in the *Crown Lands Management Bill 2016* the Government contemplates vesting Crown land subject to native title rights and interests to Local Government. In undertaking this vesting, the Government proposes, with the consent of the Local Government Authority, to transfer the liability for Local Government acts.

3.3 Dealing in Land Where Native Title Applies

The *Native Title Act* allows for 'future acts' to be undertaken on land where native title applies subject to procedural rights. A future act is a proposal to deal with land in a way that affects native title rights. A future act will be invalid where it affects native title unless it complies with the procedures set out in the *Native Title Act*. These procedures vary depending on the nature of the future act. Future acts include:

- Primary production.
- Management of water or airspace.
- Extension or renewal of pre-existing rights.
- Public housing, education and health facilities for the benefit of Aboriginal people.
- Plans of management and construction on public reserves in accordance with purpose or at least to no greater native title impact than the purpose.
- Facilities for services to the public.
- Low impact acts.
- The grant of a mining tenement.
- The compulsory acquisition of land.

The procedural rights vary based on the future act, but include the:

- Right to comment.
- Right to be consulted.
- Rights of ordinary title holder.

- Right to have an objection heard.
- Right to negotiate.

A range of outcomes and agreements can be achieved following the filing of a Native Title Determination Application, including:

- Consent determinations.
- Determinations that native title does or does not exist following litigation.
- Indigenous Land Use Agreements (ILUAs).
- Future Act Agreements.
- Memorandum of Understandings.

The impact of invalid acts is becoming increasingly prevalent. A lease issued invalidly over land subject to the rights and interests of native title may not provide lawful use and occupation for the purposes of a claim under the *Aboriginal Land Rights Act*.

3.4 Native Title Claims

A Native Title Determination Application or a Native Title Claim is an application made to the Federal Court under the *Native Title Act*. The Application seeks a Federal Court determination as to whether native title continues to exist over the land and the nature of the rights and interests held by the native title claim group.

3.4.1 Claimant Applications

A claimant application is made by a group of people, a native title claim group, who declare they hold rights and interests in an area of land and/or water according to their traditional laws and customs. The members of the native title claim group are seeking a decision from the Court that native title exists, so their rights and interests are recognised by the common law of Australia. This is called a native title determination. A determination is a decision by the Federal Court or High Court of Australia, or a recognised body, that native title either does or does not exist in relation to a particular area of land or waters (National Native Title Tribunal, 2017).

3.4.2 Non-Claimant Applications

A non-claimant application is made by a person who holds a non-native title interest in an area of land and/or water. This could be the Commonwealth or a State, or a person or organisation that holds a lease or licence. A non-claimant applicant is generally seeking a decision from the Court as to confirm that native title does not exist in relation to the area of land and/or water covered by the application.

Non-claimant applications cannot be made in areas where there is a current Native Title Determination Application or a court has already determined that native title exists. In most cases it will not be necessary for a person or an organisation making a non-claimant application to obtain a determination of native title. This is because where a non-claimant is unopposed, protection for doing the future act will apply under section 24FA of the *Native Title Act* (National Native Title Tribunal, 2017).

4 COMPENSATION AND THE PUBLIC LAND MANAGER

4.1 The Right to Compensation

Where native title has been determined to exist, the native title holders may lodge a claim for compensation for the extinguishment, impairment or suspension of their native title rights. The entitlement to compensation arises under the *Native Title Act* 1993 (Cth) for any loss and/or impairment which arises from Government acts on land that occur after the commencement of the *Racial Discrimination Act* 1975 (Cth) on 31 October 1975.

The Commonwealth is liable to pay compensation for acts attributable to it (such as the acquisition of land). The States and Territories are liable to pay compensation for acts that they are responsible for, e.g. the grant of freehold or leasehold interests in land across the State. In certain circumstances, the *Native Title Act* provides for this liability to be 'passed on' to Local Government authorities, other statutory entities and companies engaged in the resource sector and infrastructure projects. This 'passing on' occurs either through legislation or under contract.

Native title compensation is assessed by the Federal Court in accordance with section 51 of the *Native Title Act*. Under that statute, compensation is to consist of monetary payments, however, under section 51(3) the compensation claimant may request that the compensation (or part of it) is to include the transfer of property or the provision of goods and services.

While section 51A of the *Native Title Act* suggests that a potential cap is placed on the total native title compensation payable at the market value of the land concerned, the *Native Title Act* also provides that the compensation must be made on 'just terms' (sections 51A and 53). Accordingly, any extinguishment or impairment of native title under the *Native Title Act* must occur consistently with section 51(xxxi) of the Constitution. This section requires the Commonwealth to provide 'just terms' for compulsory acquisition of property. Using 'just terms' as a basis for valuation allows the court to consider a wide range of factors (not merely economic loss) when assessing the value of the native title that has been extinguished or impaired.

4.2 Griffiths v. Northern Territory of Australia (No. 3)

In August 2016, Justice Mansfield of the Federal Court delivered a determination of native title compensation in *Griffiths v. Northern Territory (No. 3)* [2016] FCA 900 (Griffiths #3). While the decision is currently on appeal to the Full Federal Court, it is significant as it is the first decision of the Federal Court to consider the methodologies to be used to value impacts on native title after the matter had been subject to a contested hearing. Prior to Griffiths #3, litigation regarding the quantum payable for native title compensation between States and native title holders was generally concluded by the negotiation of confidential agreements without the benefit of a judgement examining the detail and basis of the compromise.

Here, the case concerned the amount of compensation payable to the Ngaliwurru and Nungali Peoples, the native title holders of non-exclusive native title rights to land and waters in the remote township of Timber Creek in the Northern Territory. It had been determined in previous proceedings that the Northern Territory was liable for the acts for which compensation was sought. These 'determination acts' were attributable to the Territory within the meaning of section 239 of the *Native Title Act*.

In essence, the Federal Court ordered the Northern Territory Government to pay $3.3 million to the native title holders as compensation for the impact of land grants and public works on their non-exclusive native title. The compensation was made up of the following components:

- $512,000 for economic loss (calculated by reference to the freehold value of the land).
- $1,300,000 for non-economic loss (caused by a loss of traditional use of and connection to the land).
- $1,488,261 for interest on the economic loss component of the compensation.

4.2.1 From When is Compensation Calculated?

The *Native Title Act* does not expressly provide for the date on which the entitlement to compensation arises or the date at which the value of any interests being acquired or extinguished is to be determined (*Griffiths #3* at [117]). Justice Mansfield determined that the entitlement to compensation for an 'act' arises, for example, from the date of the grant of the freehold or leasehold interests or the construction or establishment of the public work (*Griffiths #3* at [121]).

4.2.2 Economic Loss

The component of economic loss was calculated at 80% of the freehold value of the land. Put simply, the Commonwealth had argued that 50% of the freehold value more closely represented the value of non-exclusive native title rights, if you considered that exclusive rights equated to the full beneficial entitlement to a freehold estate. However, his Honour considered that the nature of the non-exclusive rights held were closer to exclusive rights and so reached the figure of 80% on an 'intuitive' basis. His Honour's reasoning on this point is summarised in the following paragraphs [232] to [234]:

> *232 The rights the Claim Group in fact enjoyed were in a practical sense exercisable in such a way as to prevent any further activity on the land, subject to the existing tenures. If the appropriate test were as to the price at which the claim group would have been prepared to surrender their non-exclusive native title rights, the answer would be not at all. If the appropriate test was to see what was the value to the Territory of acquiring those rights, as the Territory would not then be restricted by the nature of those rights which were surrendered, the answer is that that would be a figure close to the freehold value. In my view, the appropriate valuation should be 80% of the freehold value.*
> *233 As each of the submissions recognised, that is not a decision as a matter of careful calculation. It is an intuitive decision, focusing on the nature of the rights held by the claim group which had been either extinguished or impaired by reason of the determination acts in the particular circumstances. It reflects a focus on the entitlement to just compensation for the impairment of those particular native title rights and interests which existed immediately prior to the determination acts.*
> *234 I have been careful, in reaching that intuitive figure, not to reflect in that percentage an allowance for the elements which are related to the cultural or ceremonial significance of the land, or of the very real attachment to the land which the Claim Group as an Indigenous community obviously has, and which is acknowledged by both the Territory and the Commonwealth. That is a separate and significant element of the entitlement to compensation. It is separately discussed later in these reasons. However, it is necessary to note it at this point to avoid any suggestion that the percentage arrived at represents a form of double compensation, by putting a particular value on the rights by reason of the cultural and spiritual significance of the land to the*

Claim Group.

4.2.3 Non-Economic Loss

Perhaps the most controversial aspect of the decision is the value which the judge accorded to non-economic value, that is described in paragraph [234] of *Griffiths #3* (as extracted above), as representing "the cultural or ceremonial significance of the land, … the very real attachment to the land". The compensation claimants had claimed $2 million for non-economic loss which was "to give effect to the diminution or disruption in traditional attachment to country and the loss of rights to live on, and gain spiritual and material sustenance from, the land" (*Griffiths #3* at [46]). His Honour decided that $1.3 million more closely reflected the impact on "traditional attachment".

The parties did not dispute that an award in the form of 'solatium' was appropriate in the circumstances. The judge adopted the description 'solatium' to describe the compensation component which represents the loss or diminution of connection or traditional attachment to the land, and was considered to be a suitable focus for ensuring that there is no overlap with the compensation awarded for the economic loss component.

The main issue for the Court was how to quantify "the essentially spiritual relationship which Aboriginal people, and particularly the *Ngaliwurru-Nungali* People, have with country and to translate the spiritual or religious hurt into compensation" (*Griffiths* #3 at [291]). The process of assessing this aspect of compensation was again referred to as "complex" but "intuitive" and not a "matter of science or of mathematical calculation" (*Griffiths #3* at [302] and [383]). Justice Mansfield considered the evidence of the compensation claimants regarding the impact of developments and other acts on their traditional attachment, how they had either tried to prevent the development or characterised the impact and how these acts had had a cumulative and incremental effect on the group's ability to exercise their native title rights (*Griffiths #3* at [290]-[384]).

4.2.4 Interest Calculations

Justice Mansfield considered that in the circumstances interest on the market value of the land forming part of the compensation was to be calculated on the basis of simple interest. However, the judge's comments suggest that compound interest may have been applied if instead the land had been unlawfully occupied for any period of time.

4.2.5 Invalid Acts are Compensable

Compensation was also sought in relation to three grants of freehold by the Northern Territory Government in 1998. These acts were not undertaken in compliance with the future act regime of the *Native Title Act*. Accordingly, the grants were not valid and did not extinguish native title. However, as the *Native Title Act* does not provide for compensation for invalid acts, the compensation claimants made a claim for damages for trespass (see *Griffiths #3* at [449]-[462]). The judge ordered that compensation was payable for the economic value of the non-exclusive native title rights at 80% of the land value for the three parcels, which in total amounted to $19,200 (*Griffiths #3* at [434]).

4.3 The Impact on the Public Land Manager

As mentioned, the Northern Territory Government has appealed the *Griffiths #3* decision on a

number of grounds. Whatever the outcome of this appeal, the issues are of such significance that any decision of the Full Federal Court will be appealed to the High Court. Only then will there be greater certainty about the principles to be applied when valuing native title for the purpose of compensation under the *Native Title Act*.

However, the decision is important to public land managers (whether local government authorities and other statutory entities who compulsorily acquire native title or project proponent corporations and infrastructure providers) for the following key reasons:

- Councils, statutory entities and government owned corporations who compulsorily acquire native title are liable for native title compensation (section 104 of the *Native Title Act* 1994 (NSW)).
- Liability for native title compensation is also 'passed on' by State Government to third parties in the mining context (e.g. section 281B of the *Mining Act* 1992 (NSW)) or to infrastructure providers and major project proponents via contractual arrangements (e.g. by the imposition of conditions of long-term leases).
- Most recently, Division 8.4 of the *Crown Lands Management Bill* 2016 introduced provisions seeking to create compensation responsibilities concerning native title rights and interests for the conduct of Crown land managers and local councils.

5 NATIVE TITLE AND THE SURVEYOR

As discussed in section 3, the *Native Title Act* aims to address the historical dispossession of Aboriginal and Torres Strait Islander peoples from their land by providing recognition of traditional connection to land. Accordingly, the Court, when considering the issue of extinguishment of native title, requires any party asserting extinguishment of native title to provide a high standard of evidence to establish that extinguishment has taken place. This evidence is generally in the form of government gazette notices or other official documents in the form of dealings or plans. These documents establish the legislative authority for the particular act which affects native title, the legal person responsible for the act, the date it was done and the precise extent of land or waters covered.

5.1 Doyle on behalf of the Iman People #2 v. State of Queensland

This judgement by Justice Reeves considers whether an area of land identified as a road on a plan of that parcel of land has extinguished native title as well as questions of whether native title has been extinguished by various types of leasehold grant in respect of a handful of other lots. Under section 23B(7) of the *Native Title Act*, a "public work" that is established on or before 23 December 1996 will be a "previous exclusive possession act" (i.e. it is an act that extinguishes native title). The term "public work" includes a road that is "established by or on behalf of the Crown" as defined in s 253 of the *Native Title Act*. The main aspect in dispute in this case was whether the road in question was established (or dedicated) by the Crown.

The applicant accepted that the Full Court decision in *Fourmile v. Selpam Pty Ltd* (1998) 80 FCR 151 (*Fourmile* case) was binding authority to the effect that, where land is surveyed and declared open for selection in accordance with the statutory process prescribed under a relevant piece of legislation in force at a particular time (in this case the *Land Act* 1897) and the official survey plans prepared in following that process showed an area of land marked as a road, that combination of factors was sufficient to constitute the dedication of that area of land as a public road.

The applicant also accepted that, if there was evidence that a survey was performed and a plan was prepared and charted in accordance with section 77 of the *Land Act* 1897, and if that plan showed an area of land set aside as a road, and if there is evidence that a proclamation was subsequently made under s 75 of that Act that the land so surveyed was declared open for selection, that *series of steps had the effect of dedicating that area of land as a public road*. However, the applicant contended that certain of these steps were not evidenced so could not be said to have occurred and further, they disputed the location of the road boundary.

Interestingly, Justice Reeves devoted paragraphs [8] to [44] of his judgement to examining whether these steps were in fact undertaken and the legal requirements satisfied. Importantly, he considered the legislative regime for creating roads in force at the time the road was created, namely the *Land Act* 1897 (Qld). His Honour also placed great reliance on the affidavit and oral evidence under cross examination of Mr McClelland, a principal surveyor for the State of Queensland, with approximately 40 years' experience as a surveyor, including 33 years employed as an officer of the Department of Natural Resources and Mines and its predecessors (*Doyle* at [6]). In his written and oral evidence, Mr McClelland described, among other things, the surveying and mapping practices followed in Queensland dating back to the late 19th and early 20th centuries.

A number of provisions under the *Land Act* 1897 (Qld) must be read together to understand the precise legal requirements to be satisfied to validly create a road. For example, his Honour considered the requirements for surveyed and unsurveyed land where land is proclaimed open for selection, and the other legislative preconditions to conveyance, or the creation of a road. He also noted that a plan indicating certain details was to be created and placed on public exhibit at the relevant departmental office (*Doyle* at [10]).

The plan shown in Figure 6 was extracted and discussed in the judgement with the following note at [11]:

> *... The particular part of area 33 that is at the heart of these road issues is a swamp area which is located in about the middle of the western boundary of Portion 11v on Plan LAB4012 and Portion 26 on Plan LE9. The southern part of Portion 11v was subdivided to become Portion 26. The following is an enlarged view of Plan LE9 showing the particular part of Portion 26 that is in contention in these road issues.*

To assist, the judge reproduced in his decision the detail provided by Mr McClelland regarding the information recorded on the plans and how that information is to be interpreted. The judge also extracted at length the descriptions provided in Mr McClelland's affidavit regarding the practice of the Queensland Government agencies responsible for the performance of land surveys, the preparation of survey plans and the charting and recording of those plans.

The judge accepted the State's argument that although the State could not prove that all steps were taken to create the road, in the circumstances the presumption of regularity applied. The presumption of regularity was described at [28] as follows:

> *Where a public official or authority purports to exercise a power or to do an act in the course of his or its duties, a presumption arises that all conditions necessary to the exercise of that power or the doing of that act have been fulfilled.*

In accepting the application of this presumption, the judge made close reference to the context in which the gaps occurred, noting the detailed information provided by the surveyor and the absence of any evidence provided by the applicant that the steps did not occur. The judge also accepted the evidence of the surveyor as persuasive regarding the State's position on the boundary issue. To date no NSW cases have considered the effect on native title of the dedication or vesting of roads created under the *Roads Act* 1993 (and its predecessors).

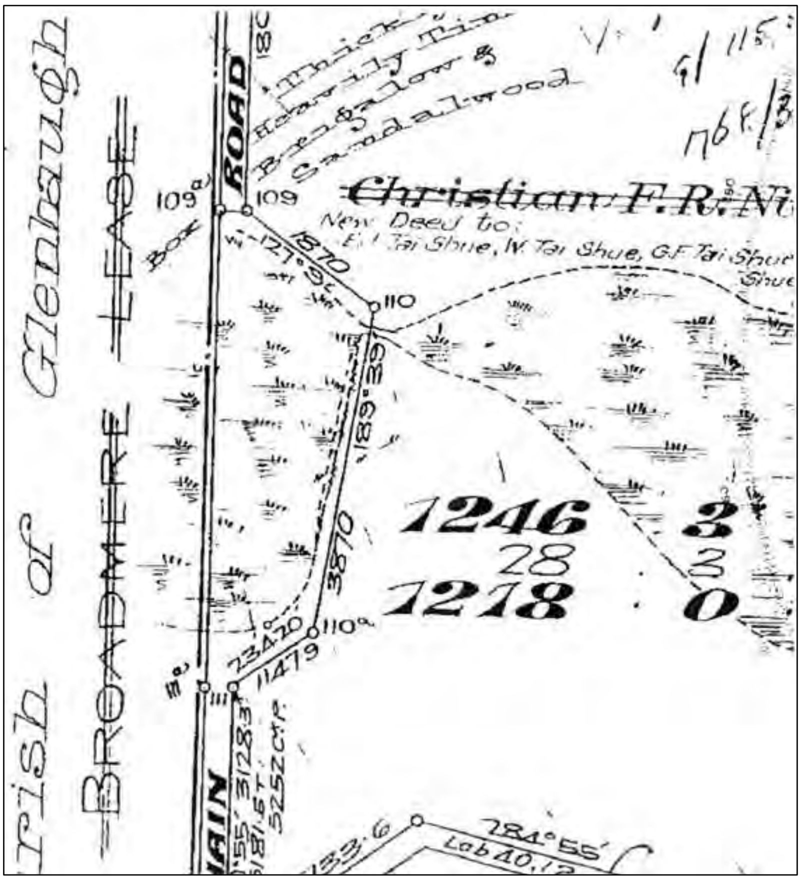

Figure 6: Enlarged view of Plan LE9 showing the particular part of Portion 26 that was in contention in *Doyle*.

6 CONCLUDING REMARKS

This paper has shown that the need to understand and address native title and native title compensation is becoming increasingly important to public land managers. The next wave of legal decisions around compensation for loss of native title rights has been commenced with a recent judgement in the Northern Territory. The financial impact of decisions by public land managers will likely be seen with increasing clarity. Surveyors, in particular, have a potentially important role in providing expert evidence and assistance in native title proceedings that require plans to be interpreted in the context of determining the extinguishing effect of a particular act which is evidenced on the plan.

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Rural Cadastral Survey of Crown Land for Aboriginal Land Grant under the Aboriginal Land Rights Act

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ABSTRACT

In 2015, the Department of Industry-Lands (DoI-Lands) issued a Request for Tender for a cadastral survey of nine titles, encompassing approximately 750 ha of land covering the locations of Mount White, Glenworth Valley, Wendoree Park and Marlow. This is the largest cadastral survey DoI-Lands has offered for open tender in recent history and represents a new challenge for the private and public sector in the management and delivery of such a large rural survey. The survey ranges from water level at the Hawkesbury River to the Mount White Trigonometric Station at RL 286 m AHD. Similarly, the perimeter of the survey extends approximately 42 km from near Mangrove Creek at the west across to the M1 Motorway in the east. The survey was commissioned for the purpose of preparing titles suitable for the transfer of land to the Darkinjung Local Aboriginal Land Council (DLALC). DLALC had claimed the land in accordance with section 36 of the Aboriginal Lands Rights Act in April 1996 and was subsequently granted the majority of the claim in December 2002 by the then Minister for Land and Water Conservation. The survey required a comprehensive and strategic approach to the management of Work Health and Safety (WHS), ranging from risks associated with working near water, quarries, farmland and horse studs, adventure and caravan parks, cliffs, bushfire and wildlife including snakes, tics, leeches and goannas. This required a proactive management of risks and work methods. It also represented challenges in locating survey marks that were placed many years ago in high-risk locations when WHS standards were less stringent. The age of existing surveys ranged from the most recent being 2006 to the oldest of 1836. The survey standards of the day and various methods of marking land, combined with the inherent challenges of the order of accuracy, particularly over steep and precipitous land, also led to significant challenges in obtaining reliable comparisons to earlier surveys. More recently, particularly in light of the amendments to the National Parks and Wildlife Act, it is inherent upon all persons to make themselves aware of the presence of Aboriginal heritage and artefacts in their location. Given the relatively undisturbed nature of the landscape, it was considered highly likely that heritage would be prevalent within the region. ADW Johnson surveyors undertook specialised training with DLALC's Cultural Officer in order to understand the likely evidence and artefacts that may be found. Ultimately, through the course of the survey, examples were found of artwork, grinding grooves and sharpening stones, which influenced the methodology applied to both traversing and marking the project. Key outcomes for our team included an entrenched appreciation for WHS issues and strategies for minimising risk, improved understanding of Aboriginal culture and heritage, and significant improvement in professional experience and competence of our staff.

KEYWORDS: *Rural cadastral survey, Aboriginal land grant, Aboriginal heritage, WHS, Mount White*.

1 INTRODUCTION

The aim of this paper is to discuss the purpose for, and the results of, the survey of Crown land being located at Mount White and surrounding areas, particularly in relation to the evolution of surveying standards confronted throughout this project. The requirement for the survey originated from a successful claim for Crown land in accordance with section 36 of the Aboriginal Land Rights Act 1983. As a result of the grant, it was necessary to complete a survey of the land to accurately determine the location and extent of the abuttals and remaining available land.

Crown land, being that land still vested in Her Majesty in the State of New South Wales (NSW), may be claimable under section 36 of the Aboriginal Land Rights Act 1983 (ALRA). Section 36 partly states:

"Claims to Crown Lands

(1) In this section, except in so far as the context or subject-matter otherwise indicates or requires:

"claimable Crown land" means lands vested in Her Majesty that, when a claim is made for the lands under the Division:

(a) are able to be lawfully sold or leased, or are reserved or dedicated for any purpose, under the Crown Lands Consolidation Act 1913 or the Western Lands Act 1901,

(b) are not lawfully used or occupied,

(b1) do not comprise lands which, in the opinion of a Crown Lands Minister, are needed or are likely to be needed as residential lands,

(c) Are not needed, nor likely to be needed, for an essential public purpose, and

(d) do not comprise lands that are the subject of an application for a determination of native title (other than a non-claimant application that is an unopposed application) that has been registered in accordance with the Commonwealth Native Title Act, and

(e) do not comprise lands that are the subject of an approved determination of native title (within the meaning of the Commonwealth Native Title Act) (other than an approved determination that no native title exists in these lands."

The ALRA was established in order to compensate Aboriginal people for the loss of land. Indeed, the preamble to the ALRA states (www.oralra.nsw.gov.au/alrareviewpreamble.html):

(1) Land in the State of New South Wales was traditionally owned and occupied by Aborigines;

(2) Land is of spiritual, social, cultural and economic importance to Aborigines;

(3) It is fitting to acknowledge the importance which land has for Aborigines and the need of Aborigines for land;

(4) It is accepted that as a result of past Government decisions the amount of land set aside for Aborigines has been progressively reduced without compensation.

Crown land can be claimed by the NSW Aboriginal Land Council, or by Local Aboriginal Land Councils, of which there are 120 across NSW. To avoid confusion, it is noted that land claimed under the ALRA is distinct from that claimed under the Commonwealth Native Title Act 1994, although land granted under ALRA may still be available for Native Title claim.

In accordance with ALRA, Darkinjung Local Aboriginal Land Council (DLALC) made a claim in 1996 to land in the locations of Mount White, Wendoree Park and Glenworth Valley. Following a detailed land and tenure review by the District Office and Land Claims Unit of the Department of Industry-Lands, a recommendation to, and approval by the then Minister

for Land and Water Conservation, the majority of this claim was granted to DLALC in 2002. Some of the land, such as roads, easement sites and privately owned land was refused. Figure 1 shows the general extents of the granted lands.

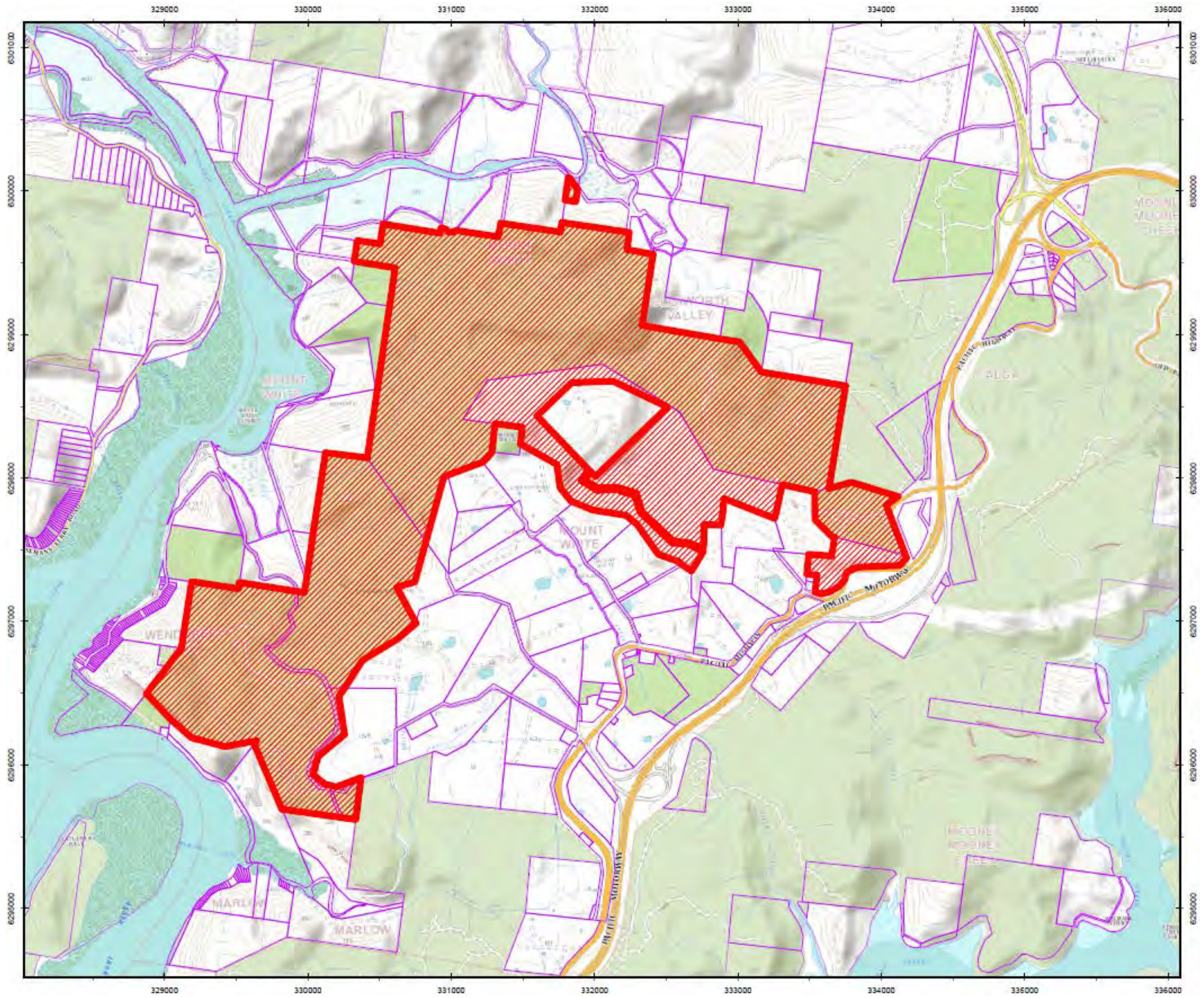


Figure 1: Aboriginal land claim 5831, granted lands.

Broadly, the land is bounded by:

- To the south, private rural land holdings and the Hawkesbury River.
- To the west, private rural holdings fronting Mangrove Creek.
- To the north, private rural holdings fronting Popran Creek and Kelly's Creek.
- To the east, Popran National Park and the Pacific Highway.

In December of 2015, the Department of Industry-Lands (DoI-Lands) offered the survey to public tender to any NSW Registered Land Surveyor. Following the assessment of a number of submissions, in April 2016 ADW Johnson Pty Ltd was tasked with completing the survey which included the granted land and several residue parcels.

The survey was to be lodged in a format suitable for registration at Land and Property Information (LPI) by May 2017 (DFSI Spatial Services, 2016a, 2016b). This timing needed to allow for the review and approval of the survey and mean high water definition by DoI-Lands prior to 16 May 2017.

2 PRE-SURVEY PREPARATION

When planning to undertake any survey, a number of common factors arise that need to be considered. These include factors such as Work Health and Safety, resourcing, management, reporting and methodology. As with most professional practices, systems are developed in order to recognise and manage repetitive risks, processes and arrangements with a view to provide a service of the highest standard, quality and efficiency. The reader is referred to the appendix for a copy of the Plan of Subdivision, including lots 1 to 15 referred to herein.

2.1 Risk Assessment

In this case, aspects of the survey such as the work environment, timeframe and risks were outside our standard operating procedures. Consequently, site specific risk assessments, and processes were developed prior to commencing the site work. Our risk assessment identified a number of site specific issues that required further consideration.

2.1.1 Work Health and Safety

Terrain: The region is mountainous and there are several areas where there are significant cliff escarpments (100 m or higher) combined with smaller drop offs, often not visible due to the thick vegetation. Consequently, it was necessary for all personnel to be well aware of their surroundings, maintaining a safe working distance from cliff faces and drop offs, whilst ensuring fatigue is managed to avoid heightened risk due to a lack of awareness (Figures 2 & 3).

Figure 2: View of the escarpment along the northern boundary of lot 14, from Hawkesbury River bank.

Wildlife: Whilst Australia is known for its deadly wildlife such as sharks, spiders and snakes, this is not always a major factor in the surveying environment. However, in this case, it was common knowledge to locals (and regularly reinforced) that the area was known for its snake population, particularly death adders (particularly around higher escarpments such as Mt White trig station), black, red belly and brown snakes. Again, alertness is a key element in the avoidance of this risk, and several were encountered throughout the survey without incident.

To manage this risk, first aid kits were carried with field crews at all times. The key treatment for snake bite is to apply a compression bandage, minimise movement and seek medical help urgently. In isolated regions of the site, the only effective means to comply with this requirement is to evacuate personnel by helicopter. Consequently, all crews carried personal EPIRB beacons which could be activated should this need arise.

Figure 3: (a) View from the southernmost corner of lot 10 south to Marlow and the Hawkesbury River, and (b) access to TS4746 Mount White.

Isolation: EPIRBs were also essential for managing other risks such as bushfire and falls. Some of the site had no mobile phone reception so it was essential that personnel could be located accurately and evacuated if required. Management also remained in contact with the local Rural Fire Service control centre and National Parks and Wildlife Service (NPWS) so that the risk of bushfire and controlled burning could be managed.

Chainsaw use: As this is a rural survey, many of the marks and reference marks were trees, particularly in the northern and western sector of the project. In accordance with the Surveying and Spatial Information Regulation, it is also necessary to clear and blaze the boundary lines in the course of completing the survey. Consequently, it was determined that chainsaw use would be a regular requirement during the course of the survey to expose reference tree shields (Figure 4) and also increase the speed with which boundaries could be cleared and blazed.

The use of chainsaws required third-party training to be completed by three of our personnel. New chainsaws were also purchased along with necessary safety and maintenance equipment. Ultimately, fuel efficiency, reliability and (most importantly) weight were factors which determined the model of chainsaw to be used.

Adjoining land use: The conduct of the field survey also needed to consider the impact on adjoining land owners and particularly their business operations. Adjoining land use is varied, ranging from simple rural residence, through to residential and holiday caravan parks, sandstone quarry, adventure park, equine industry and grazing.

Figure 4: Opening a shield (before and after).

Each situation required cooperation and flexibility on the surveyors' part in order to minimise impacts, whilst operating as efficiently as possible. Risks and issues included upset residents and visitors, injuring livestock (some very expensive), danger from operating machinery, injuring bushwalkers and inexperienced horse riders (Figures 5 & 6). Some arrangements also took weeks or months to coordinate, such as tracking down an owner living in China or operating in certain areas only during school terms.

Figure 5: Stock at the Glenworth Valley Adventure Park.

Figure 6: Machinery at the Gosford Quarries Mt White facility (Mt White in the background).

2.1.2 Aboriginal Heritage

In accordance with the National Parks and Wildlife Act, it is inherent upon all persons to make themselves aware of the presence of Aboriginal heritage and artefacts in their location (Department of Environment, Climate Change & Water, 2010). Given the nature of the survey was for an Aboriginal land grant (creating some heightened awareness of this issue) and the relatively undisturbed nature of the terrain, it was considered likely that Aboriginal heritage could be encountered in the course of completing the survey. It was therefore arranged for personnel to complete training with the DLALC heritage officer. This training demonstrated some of the types of artefacts and features likely to be encountered, combined with an understanding of the areas of the landscape which were more likely to contain heritage items.

As a part of this training, it was recognised that the placement of survey marks in rock escarpments, whether they be stations, boundary marks or reference marks, may be damaging Aboriginal heritage. Often rock escarpments may be obstructed with ground coverings so it is not possible to know what artefacts are in the area. Our training demonstrated that, even where specific engravings or carvings may not be affected, the placing of survey marks in close proximity to artefacts, such as on the same rock shelf, may be offensive to Aboriginal culture. Consequently, it was necessary to modify our survey traverse and marking methods in order to minimise the risk of damaging heritage items. This included not placing survey marks in large sandstone escarpments (suitable for engraving), particularly in the elevated regions of exposed rocky outcrops. Other items to consider included middens (collections of crustation shells), rock arrangements (such as around campfires), grinding stones and grooves, sharpening stones and spearheads. Residents were also aware of many locally known (although undocumented) heritage sites.

Whilst heritage items were speculated to have been identified and avoided throughout the course of the project, Aboriginal heritage was identified and reported at only one location, near the south-eastern corner of lot 6. This heritage was a cave which was located only metres from the corner (Figure 7). Once identified, the area was avoided and a report, location and photos sent to the NPWS and DLALC. Whilst it is not always possible for the untrained eye to be certain of the authenticity of potential artefacts encountered throughout the course of the survey, our awareness of the types of artefacts, combined with a policy of extreme caution and avoidance in all situations, minimised the risk of any impacts on Aboriginal heritage.

2.2 Resources

In order to meet the project timeframe, it was determined that it would be necessary to allocate, on average, at least 1.5 field crews per week to the project, i.e. 7.5 days per working week. Whilst this was quite manageable, it also meant being sensible about other opportunities and projects which should (or should not) be pursued by the company in order to maintain timeframes and service standards to existing commitments.

Due to the unique nature of the project, it was also considered an opportunity for professional development of key personnel with the company to obtain experience and training. Consequently, two of the company's long-serving graduate surveyors were allocated to the project, combined with the most experienced technical surveyor, to deliver the majority of the field work (under supervision). Consequently, one of the graduates is now using the survey of lots 9-14 (south of Morgans Road) as his rural project for the Board of Surveying and Spatial Information (BOSSI) candidate assessment.

Figure 7: (a) Cave containing Aboriginal drawings, and (b) charcoal drawings within the cave.

2.3 Management

The unique nature of the project required management systems to be implemented to ensure the project would operate as efficiently and effectively as possible. This included matters such as on-site monitoring of field work as appropriate, review of field notes, electronic data and record keeping on a weekly or shorter interval (as appropriate) and management of workflows on a weekly basis. Other factors also included the extent to which investigations were made on site for the location of survey marks. To this end, it was necessary to ensure all avenues and possible scenarios and calculations were considered prior to consider classifying marks as 'gone', 'destroyed' or 'not found'.

Consultation with adjoining land owners was also a unique element of the project as it was often necessary to obtain access through adjoining owners' property to the Crown land, whilst also garnering local knowledge of risks and site constraints. Ultimately, there were 21 different adjoining owners to liaise with and this was a key management role for the registered surveyor, in case it was necessary to invoke the 'Power of Entry' rights under the Surveying and Spatial Information Act. Ultimately this was not required, although in some cases as properties were not regularly occupied, it took some weeks or months to contact owners to arrange access.

3 BOUNDARY DEFINITION

With regard to the Plan of Subdivision in the appendix, the purposes of the lots within the plan are:

- Lots 1, 2, 5, 7, 8, 9, 10, 11, 12: Land to be transferred to Darkinjung LALC in accordance with Ministerial Grant of part of ALC 5831.
- Lots 3, 4, 6, 13, 14 and 15: Land within the subject parent lots not granted to Darkinjung LALC and to be retained by the Crown (residue lots).

Whilst the survey covered some 42 km of boundary marking and 58 km of traversing, this section focuses on the areas covering the older and more challenging aspects of the boundary redefinition. This incorporates the northern boundaries of the survey, generally from the junction of Mangrove and Popran Creeks, east across to Popran National Park, including lots 6 & 7 and the northern boundaries of lot 5 (Figure 8). The remaining areas of the survey, whilst significant in length and area, were generally defined by modern surveys of the 20th and 21st century and the definition is explained clearly within the Plan.

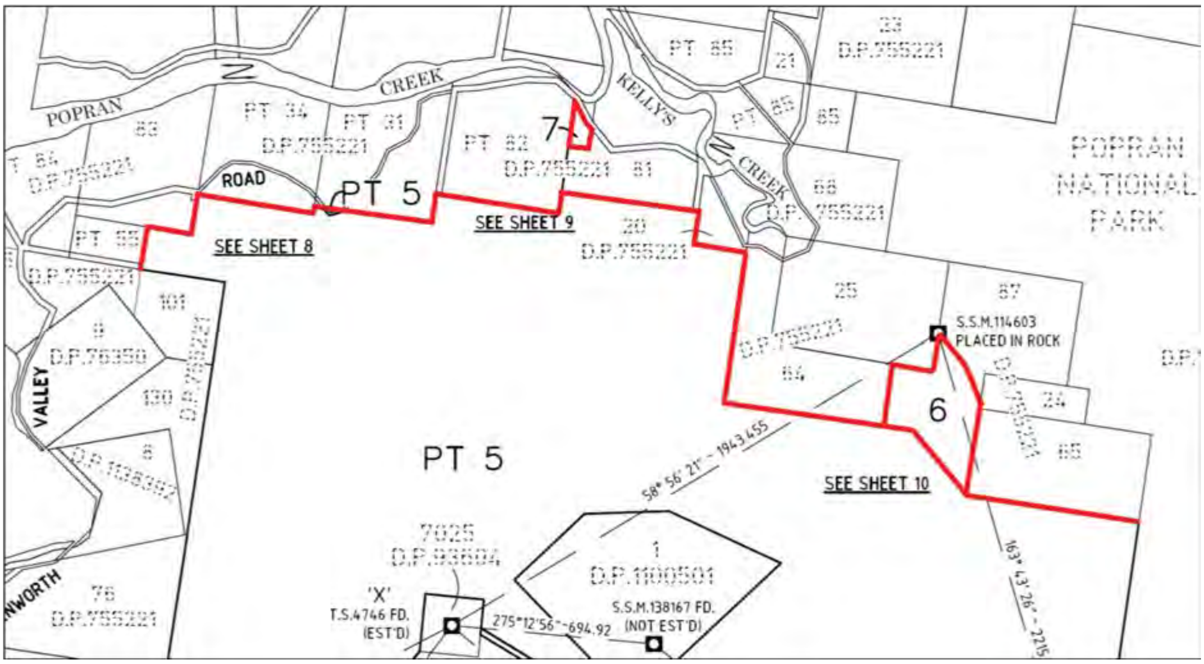


Figure 8: Area of boundary subject of this section highlighted in red.

Broadly, the field survey commenced at the Pacific Highway adjacent to lot 2, proceeding clockwise via TS4746 Mount White, south to Morgans Road, then north-west along Morgans Road and then northerly along the western boundary. It was considered less efficient to progress anticlockwise, as the boundary between lot 5 and 45/1197008 (Popran National Park) had never been surveyed and was only created by a simplistic description in the simplistic reserve gazettal. That boundary was ultimately agreed by the Crown and NPWS to be an extension of the eastern boundary of portion 65, southward to lot 1. A separate loop was undertaken for the definition of lots 9 to 14 inclusive, south of Morgans Road.

3.1 Northern Boundary – Stage 1: Ph 101, 55, 84 and 83

Initially, as had been the approach for the entire survey, the aim was to traverse the abuttal boundary with a view to finding marks that had been placed in the original adjoining surveys. This approach allowed for the hierarchy of marks, particularly in relation to Crown grant surveys and occupations, and had been relatively successful for the earlier survey.

Leading up to this, during the course of the definition of the western boundary, several reliable original marks had been found which defined adjoining portions 76, 101 and 130. A traverse connection had also been completed from the eastern boundary of portion 76, across to control on Mount White trig station. Consequently, it was considered that the calculations north and eastward from portion 55 and 101 were reliable.

Having regard to Figure 9, during the course of the abuttal traverse, original marks were found at corners A, B and C. Corner A (Figure 10) had been placed in the course of the survey for portion 101 (N4505, circa 1910) whilst at corner B (Figure 11), the original corner mark is noted as found from Ph 55 (N3824, c. 1903) and the reference tree was created in N4505. Corner C (Figure 12) had been placed for the survey of Ph 83 (N1021, c. 1883). Distance A-B is per original (PO).

In the absence of a mark being found at corner D, it was necessary to confirm the abuttal by means of extending the survey to corners E and F. The distance between corner marks found at E and F is 0.15 m in excess of N1021 (Ph 84) and 0.55 m in excess of N3824 (Ph 55). Distance C-E was 0.555 m short of PO vide N1021 (Ph 83).

Due to the shortage of line C-E and in order to confirm the position of corner D, the survey was extended southwards to corner G where a peg and reference tree were found. It should be noted that the tree was originally referenced in N3824 (Ph 55), however this was re-referenced in N4505 (Ph 101) due to what appears to be a typographical error (typographical or transcription) of approximately 30º. The reference changed from 159º00’ 54 lks. to 189º45’ 53.5 lks.

Whilst the original intention, as represented in N3824 is for line D-G to be straight, due to discrepancies in both angles and dimensions between plans N3824 and N4505, it was decided to maintain the abuttal dimensions of line D-E as PO of 247.435 m and create a bend at corner B. It is also noted that N1021 was modified to include a subsequent diagram showing different bearings to the original survey. Indeed, the original plan drawing (Figure 13) was embellished with new (non-cardinal) bearings in red, along with the diagram. These details are generally consistent with plan N3824 of Ph 55.

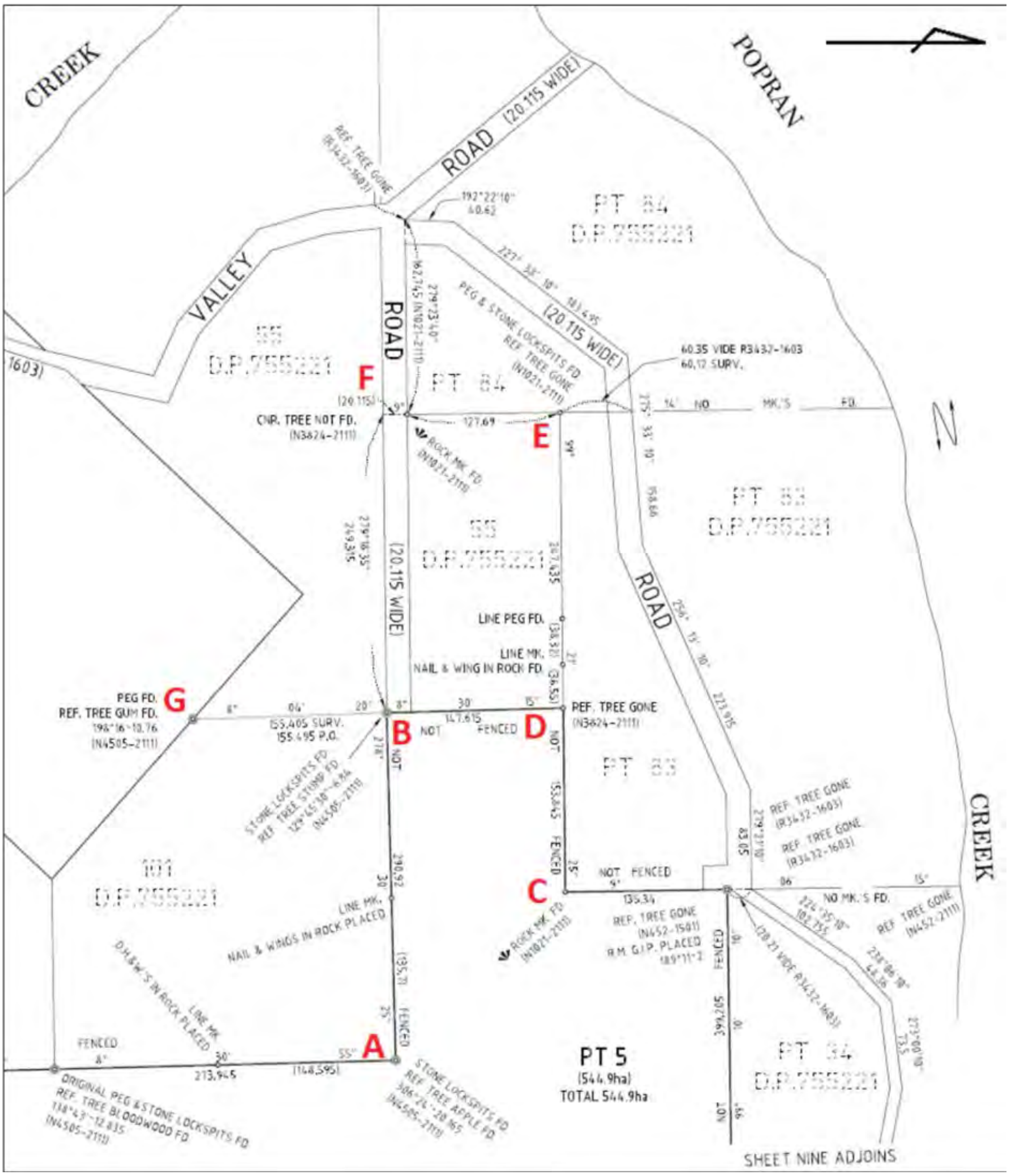


Figure 9: Extract of page 8 of the Plan of Subdivision.

Figure 10: Corner A, c. 1910.

Figure 11: Corner B, c. 1903.

Figure 12: Corner C, c. 1883.

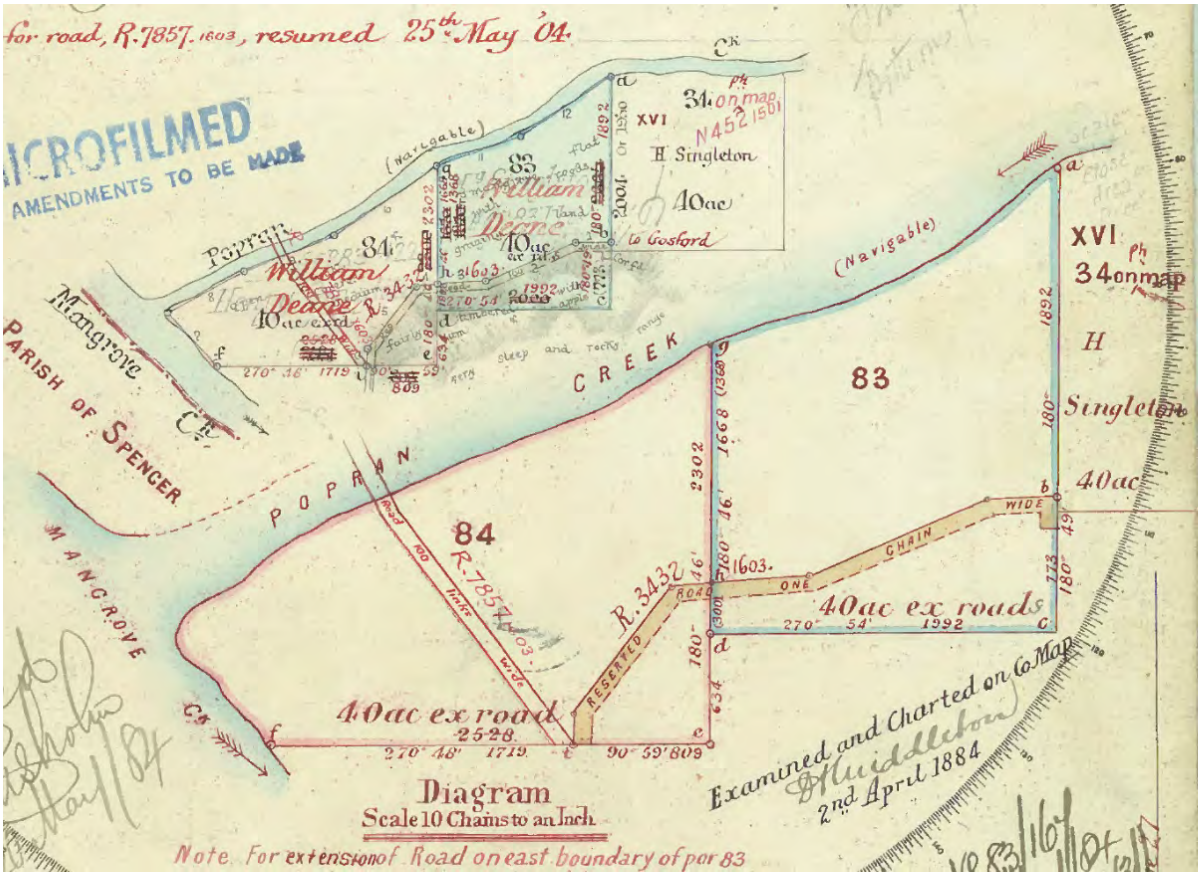


Figure 13: N1021, Ph 83 and 84, extract.

3.2 Northern Boundary – Stage 2: Ph 83, 34, 31 and 82

3.2.1 Historical Survey Practice

Ph 31 and 34 were two of the older plans associated with the survey. At this point, it is important to recognise the different practices of surveyors throughout various iterations of directions and circulars to surveyors. Indeed, the 19th century was a formative time in the surveying profession in the Colony of New South Wales and there were significant events that defined the conduct of surveyors, many of which remain in force today.

The period encompassing these two plans was particularly significant as noted in Marshall (2002, p.13): "*The period between 1833 and 1864 is critical in the development of survey practice in New South Wales. Prior to 1836 very little appears to have been formalised regarding survey directions and the earliest directions so far found are in the form of circulars to improve marking and identification of errors.*"

Plans N397 (Ph 31) and N452 (Ph 34) were both completed by Licenced Surveyor Commins. Indeed, the plans show they were both transmitted to the Surveyor General of the day (Mr Walker Davidson) on the same date being 1 January 1863. Of course, this day represents a significant milestone in land tenure in NSW, being the day that the Real Property Act commenced and the Torrens Title system was introduced – a system that remains in place today and is recognised as one of the most effective land tenure models throughout the world.

Indeed, the most recent direction prior to the 1862 surveys was on 9 July 1853 by Acting Deputy Surveyor General Mr J. Thompson (Marshall, 2002, p.19). This direction was entitled "*Instructions for Marking Crown Land by Government Surveyors*" and known particularly for the introduction of the requirements for blazing of lines (to a width of 3 feet), lock-spitting of corners (although only in plain country) and the numbering of reference trees.

This circular makes no mention of marking corners where pegs cannot be placed, such as in rock. In this instance, the prior direction of 10 April 1848 still applies, which states "*The marks in the rocks are to be broad arrows, crosses, triangles or squares, and each licenced surveyor is requested to preserve uniformity in his own marks, so that they may be easily described and recognised*" (Marshall, 2002, p.17). As a consequence, it was necessary to keep an open mind as to the type of mark that may be found, particularly where it is apparent that a peg is unlikely to have been the original corner mark.

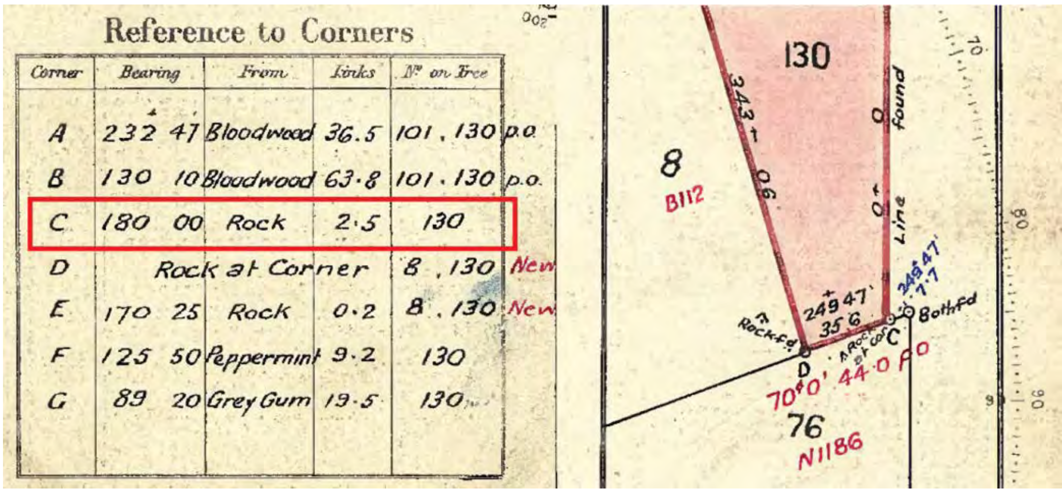
It is also important to note the referencing of Truscott (1894) in Marshall (2002, p.13), which states "*...during the period 1830 to 1850, in surveys over rough broken ground there is little likelihood of old marks being found because often in portions with river frontages extending back to cliffs, the back lines were generally not run and if the back corners were not easily accessible marking was also omitted.*" Whilst this statement applies to surveys a decade prior to the subject portions, the geography (Figure 14) and methods of measurement are similar and good practice to consider.

Another factor to be considered was that it was common practice up until about 1864 for reference marks to be quoted from the mark to the corner. It was not until after the Real Property Act of 1862, which set out requirements for the licencing of surveyors, that the first regulations were introduced in 1864 for 'Licenced Surveyors Employed by the Lands Dept.' (Marshall, 2002, p.24), showing that reference be made from the mark to the corner. In spite

of this, there was still a practice for special marks to be shown from the corner to the mark on Lands Department plans prior to 1933 (Marshall, 2002, p.14). Consequently, throughout the course of the entire survey, it was prudent to investigate references in both forward and reverse bearing directions from a corner.

Figure 14: View looking east along the southern boundary of Ph 31. The south-east corner of this portion is near the red arrow.

Figures 15 & 16 show a corner mark from Ph 130, being plan N5944. Despite the custom of chiselling the number at the corner mark, where possible, as shown in Figure 16, the portion number has been chiselled adjacent to the reference mark.



Reference to Corners

Corner	Bearing	From	Links	No. on Tree	
A	232 47	Bloodwood	36.5	101 . 130	p.o
B	130 10	Bloodwood	63·8	101 . 130	p.o.
C	180 00	Rock	2·5	130	
D	Rock at Corner			8 . 130	New
E	170 25	Rock	0·2	8 . 130	New
F	125 50	Peppermint	9·2	130	
G	89 20	Grey Gum	19·5	130	

Figure 15: Reference mark C in N5944 (Ph 130), c. 1922.

Figure 16: Reference mark C in N5944 (Ph 130), c. 1922. North is to the right as confirmed by the shadowing.

3.2.2 Site Survey

The establishment of the southern boundary of Ph 83 provided an orientation for further calculations moving clockwise through portions 83, 34, 31 and 82. Figure 17 shows the various abuttal plans, surveyors and dates of survey.

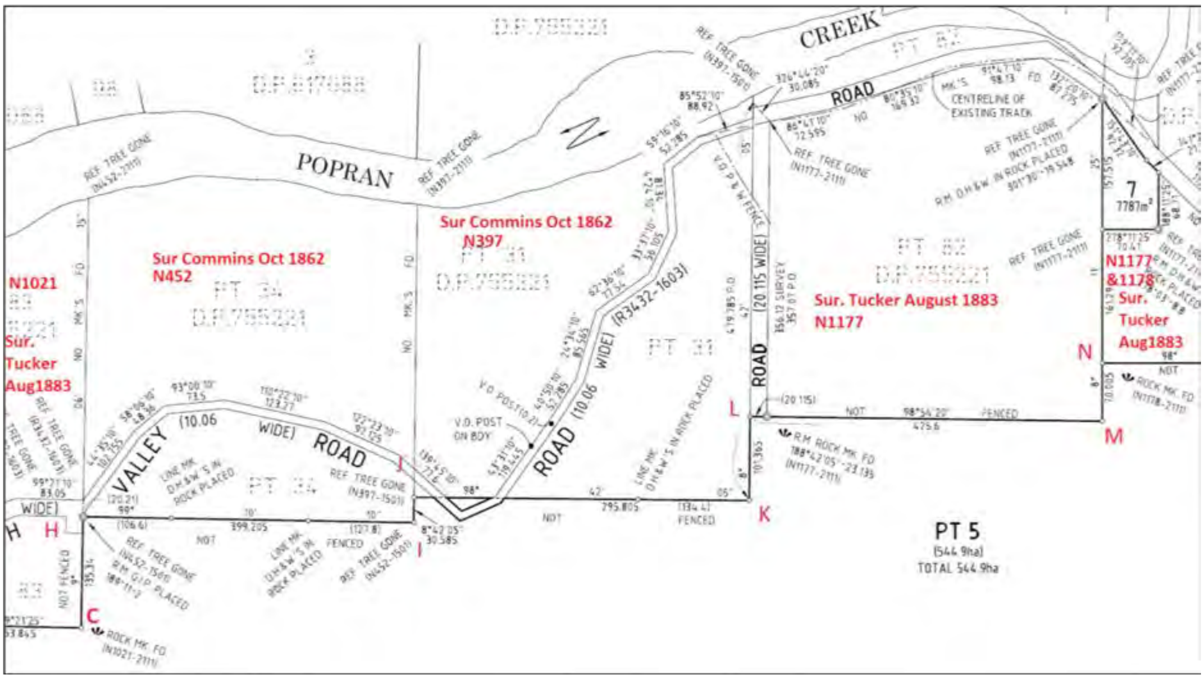


Figure 17: Extract of page 9 of the Plan of Subdivision.

Calculations were extended using original dimensions from corners C to N. The traverse was extended along these calculated boundaries from C to L. At this point no marks had been found, including the reference mark near corner L which was found later. Given the age of the surveys and terrain, it was decided to proceed northwards in order to (1) search for other marks relating to portions 31 and 34 along the reserve roads and other boundaries, and (2) close the traverse to verify the accuracy of the control. Unfortunately, no marks were found and only a couple of very old fence posts were located within Ph 31 (Figure 18).

Figure 18: Very old fence post found in Ph 31 (on the reserve road boundary).

The next course of action was to extend the traverse eastward from the north-east corner of Ph 31 along the reserve road. Again, no marks were found along the road or near the old water reserve, which is now partly represented by lot 7. The survey proceeded south along the common boundary of Ph 81 and 82 with a view to connecting back to corner L. At corner N, the corner mark broad arrow was found as shown in N1178 (Figure 19). It should be noted that Ph 81, whilst originally defined in N1177 by Surveyor Tucker, was subsequently amended in N1178 of March 1886 by Surveyor Percy Cowley affecting the southern and eastern boundaries. It was then a recalculation to corners M and L. Ultimately, the reference mark was found in the cliff face adjacent to corner L (Figure 20). No other original marks were found and, as will be explained later, the plan R3432 defining the Crown road was later used to assist with the definition of Ph 31 and 34.

Figure 19: Corner N ("on high rock") vide N1178 (Ph 82), c. 1886.

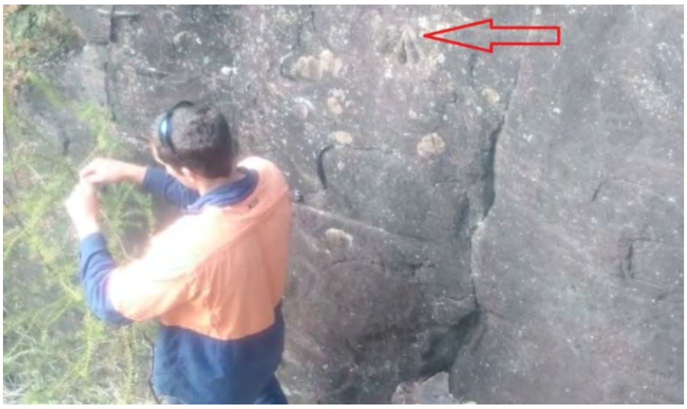


Figure 20: Reference mark adjacent to corner L vide N1177 (Ph 81), c. 1883.

3.3 Northern Boundary – Stage 3: Ph 81, 20 and 64

With the lack of success in defining Ph 34 & 31, it was decided to push forward to more modern surveys in the hope of finding other evidence that may help in back-tracking older definition. As the rock mark was found at corner N in Figure 21, further calculations were made through Ph 81 (N1178), Ph 20 (N171) and Ph 64 (N4072). Indeed, N4072 was a more recent survey from 1906 and so it was considered probable some marks would be found.

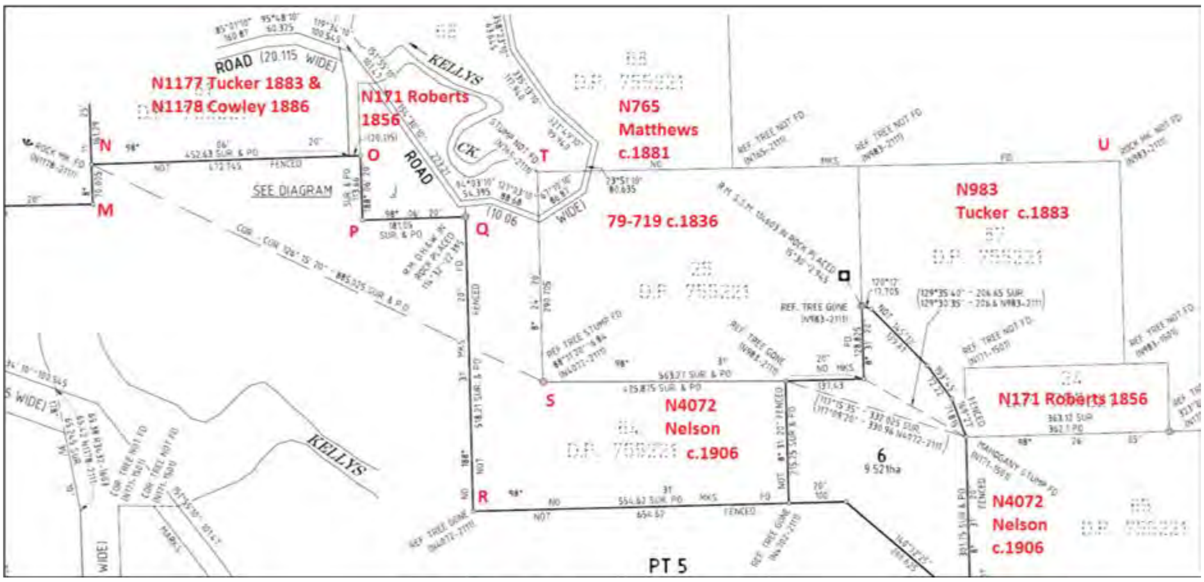


Figure 21: Extract of page 10 of the Plan of Subdivision.

The traverse was extended along the line N-O-P-Q-R with a branch off to corner S. Ultimately, no marks were found here other than a stump at corner S. With a lack of reliable connections through these various surveys at this time, it was considered insufficient evidence to be confident in the current definition. Investigations were also undertaken around the northern boundary of portions 81 and 20 as there were several reference marks, corners and road boundaries in this area. Again, no marks were found. It was likely that much of this area had been cleared and farmed over the intervening 150 years and more recently was being ploughed and seeded for agistment. It was also observed that some of the creek alignments had been diverted or channelled for farming and irrigation purposes.

Having traversed to corner R, it was also possible to see across the valley southward to the Gosford Quarry. Consequently, observations were made to connect the traverse back to the starting azimuth in order to confirm the reliability of the control. Again, as had been done along the western boundary, this ensured that the survey integrity was maintained and the connections and calculations between portions was reliable.

Following the lack of cadastral evidence found, it was considered necessary to extend the survey investigation north and east into adjoining portions in order to determine if the current calculations were reliable. This included Ph 25 (79-719), Ph 87 (N983) and Ph 68 (N765). Firstly, the survey was extended to the south-east corner of portion 25, along with the north-east corner of portion 64. Neither corner was found, nor the south-west corner of portion 87.

At this point, it was decided to investigate for other rock marks in the area which may have been more likely to last. Consequently, the traverse was extended to the boundary line T-U

with a view to trying to locate the rock mark at the north-east corner of portion 87 at corner U. A number of calculation options were undertaken, and corners investigated along this line. Ultimately, after several attempts, no marks were found, despite the relatively undisturbed nature of this country.

Of course, it has always been the custom for a new survey to connect to the adjoining surveys. To this end, each portion investigated in this survey had generally been connected to its adjoining portion, wherever they existed. However, with the varying survey standards, accuracy of measurements and observations, and different surveyors, significant variations can be found when extending across several portions. And of course, these multi-portion surveys have generally not been required until now. Consequently, it was difficult to make calculations with certainty about the location of corners of each subsequent portion, having not found any in the adjoining one. So as in this case, having only found a mark at the south-west corner of portion 81 (corner N in Figure 21), there was little confidence in calculations to marks further afield such as portions 20, 64 and 25. Having not found any on portions 68 and 87 created further doubt and a reluctance to proceed on the survey route (through to portion 65) without other information.

3.4 Northern Boundary – Stage 4: Parish Road

To the benefit of this region and this survey particularly, a road survey had been undertaken in 1887. This survey defined the parish road extending from what is now Wendoree Park (near the junction of Mangrove Creek and the Hawkesbury River), north along Mangrove Creek and extending upstream adjacent to Popran Creek to its original junction with Gosford Road, in the proximity of the Glenworth Valley Adventure Park and Riding School facilities. This included Little Mooney Mooney Creek, which is now known as Kelly's Creek. The road survey extended 7 miles, 40 chains and 13 links, being just in excess of 12 km in length. The road adjoins the subject land to the west and north throughout the abuttals discussed herein as shown in Figures 9, 17 & 21.

As required, the road survey connected to many cadastral monuments and corners throughout the course of the road. Consequently, this survey provided some comparison of azimuth throughout the valley. Mile markers and other reference marks were also placed along the survey. Part of this survey is shown in Figure 22 to which the subject boundary has been added in red.

Having been completed in 1887, this plan provided connections to corners of abuttal portions that were remaining at the time of survey. Of course, being for road purposes, it was not essential for the surveyor, Mr Finn, to define entire portions, merely the relevant boundaries as he was required and could readily define. Nevertheless, his survey connected to many of the portion marks relevant to this survey. These measurements, in many cases, either confirmed the original survey dimensions and orientation as reliable, or in a few cases, confirmed discrepancies.

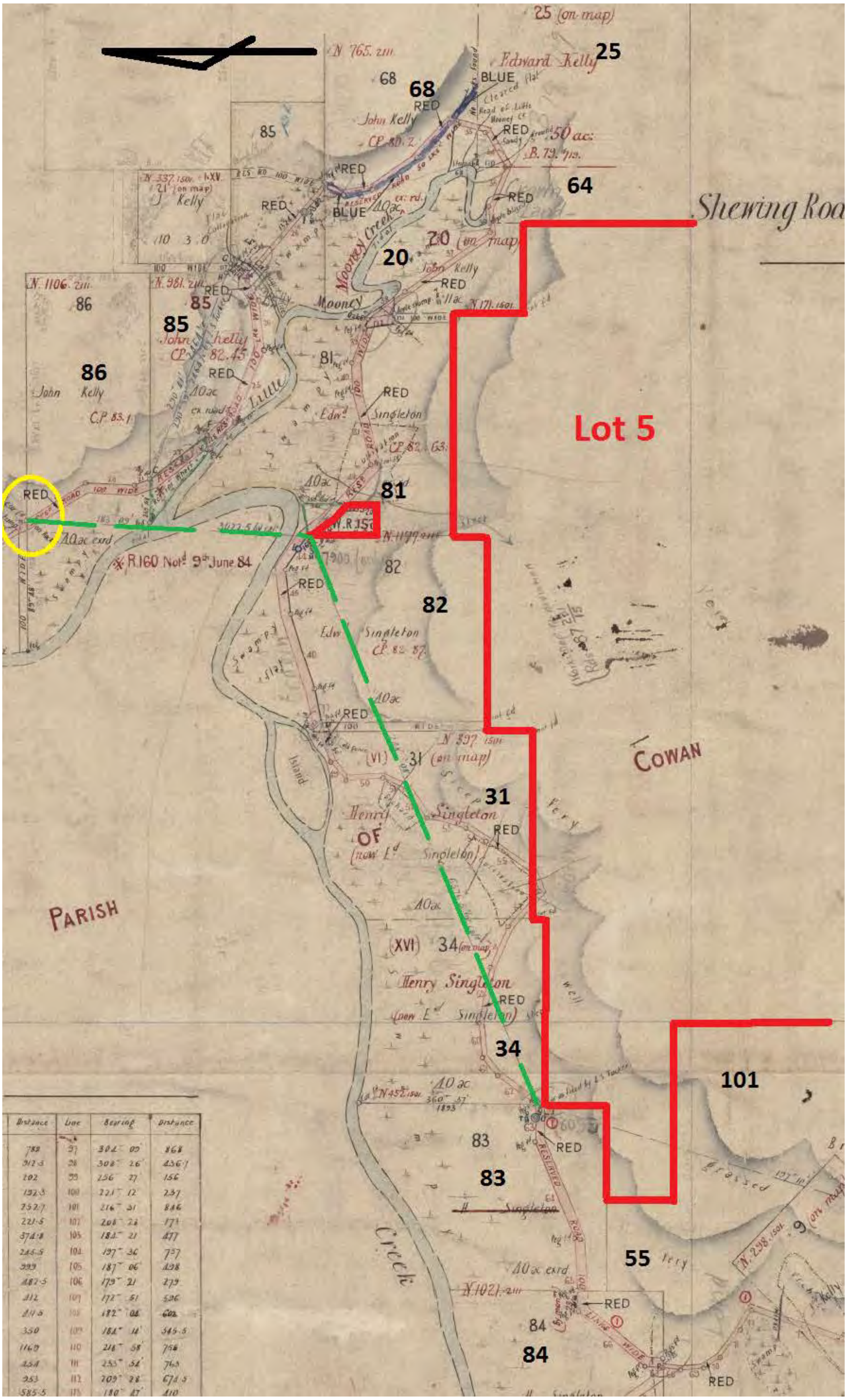


Figure 22: Extract of road plan R3432-1603.

Of course, as may have been noted by the studious reader, it was necessary to define this road, at least in part, as it extends within lot 5, cutting this lot into two parts adjacent to portion 31 (see Figure 17). With the lack of marks found from portion 83 through to portion 64, investigations were made on other plans further afield. Referring again to the road plan in Figure 22, the yellow circle shows the next nearest rock mark. This mark was placed in the original portion 86 plan, N976 of 1883, and subsequently shown in the replacement plan N1106 of 1884 (although this mark did not change). The traverse was extended to this mark following calculations via the road plan and was found as shown in Figure 23. This was a significant find for the survey definition and resulted in tightening up calculations throughout this section of the survey.

Figure 22 also shows the calculation lines from the survey in green dashed lines. These connections were to corners of relevance to the survey as it connects from the rock mark directly to the north-western corner of lot 7 and then on to the corner of portion 34.

Figure 23: Rock mark on portion road.

It was not possible to make direct comparisons on the road plan calculations, because no other marks were found. However, referring back to Figure 9, given that the line B-F had been re-established from original marks, it was decided to produce this line westerly by adopting the PO dimension in order to re-establish the road corner as shown in parish road plan R3432 and portion 84, plan N1021. Ultimately, this showed up a difference in overall length, from here to the rock mark, of 0.885 m shortage using the short line table.

Following this encouraging result, other comparisons were made. This included corner C to H (Figure 17) which showed an excess of 0.05 m to PO. Further, at corner L, calculations north to corner L1 (Figure 24) show a shortage of 0.955 m compared to PO, whilst at corner N, comparison north to corner N1 identified an excess of 2.57 m. It should also be noted that the road plan shows corner L (and K) as not found and whilst the corner mark was not found, the reference mark remains. Further, the road plan did not connect to corner N.

These comparisons give a reflection of the reliability of the earlier plans and were considered quite reliable, given the age and nature of the various surveys, and were used as a basis for further calculations and comparisons. Ultimately, this was also used for the definition of the road within lot 5 and assisted with the definition of the abuttals around portions 83, 34 and 31 in particular.

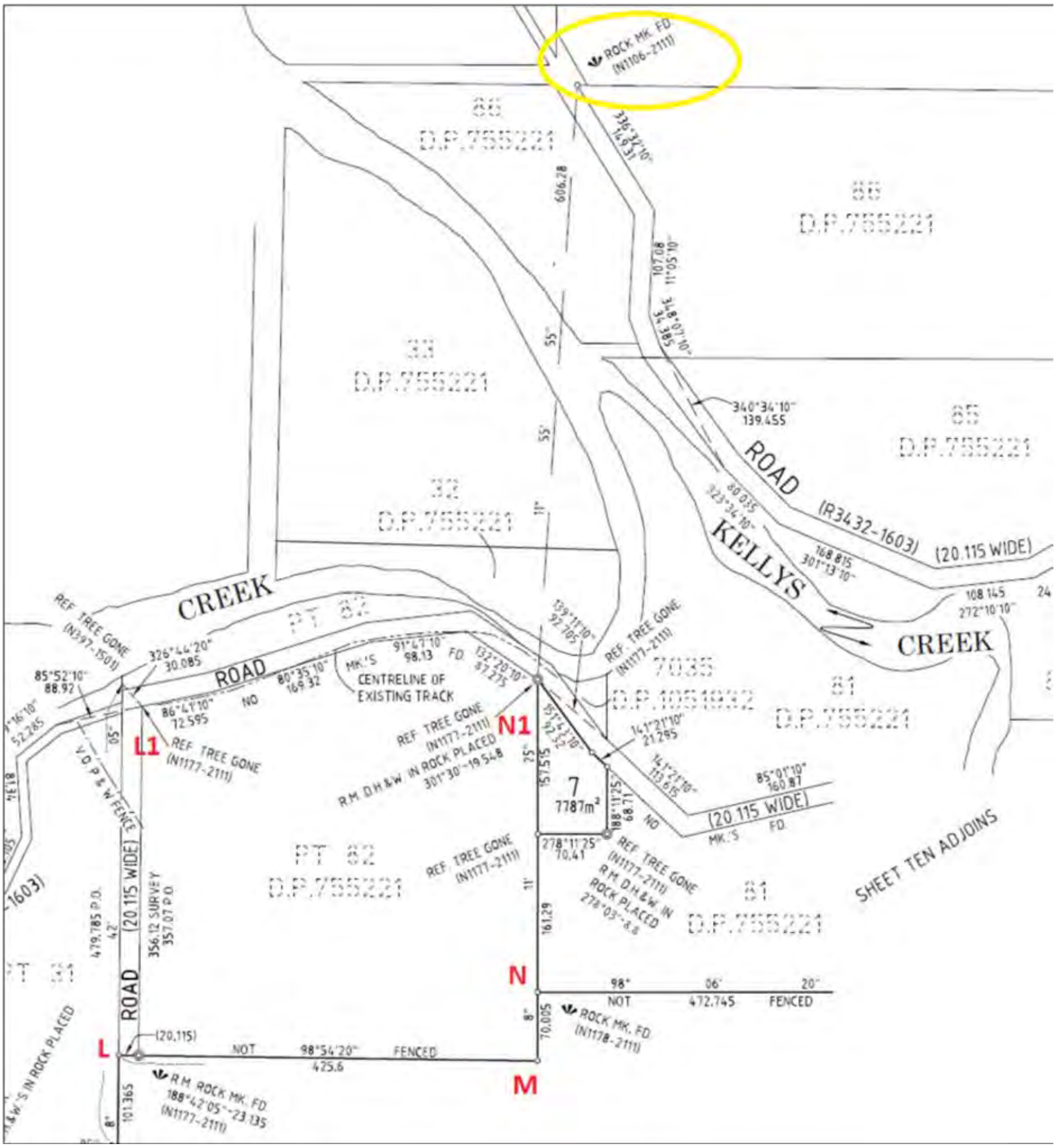


Figure 24: Extract of page 9 of the Plan of Subdivision.

Throughout the course of the investigation of marks along the road plan, the existing track was also located at appropriate sections. Some of this information has been reproduced on the Plan of Subdivision, noted as ‘Centreline of Existing Track’. The track is well established, particularly in those undisturbed regions of the survey, such as from portions 83 through to 81.

As noted earlier, fencing was also located throughout this survey and whilst it did not prove beneficial to the definition, it did exist in a position that indicated it was the same fencing shown in the road plan of 1887. Figure 25 includes an extract of part of R3432 showing fencing around a cultivated area of portion 31. Red dots on this extract indicate the approximate position of very old fence posts which were found as shown in the extracts from the Plan of Subdivision. Figure 26 shows a couple of these original posts (see also Figure 18).

This evidence combined assisted in confirming that the road redefinition was consistent with its original position.

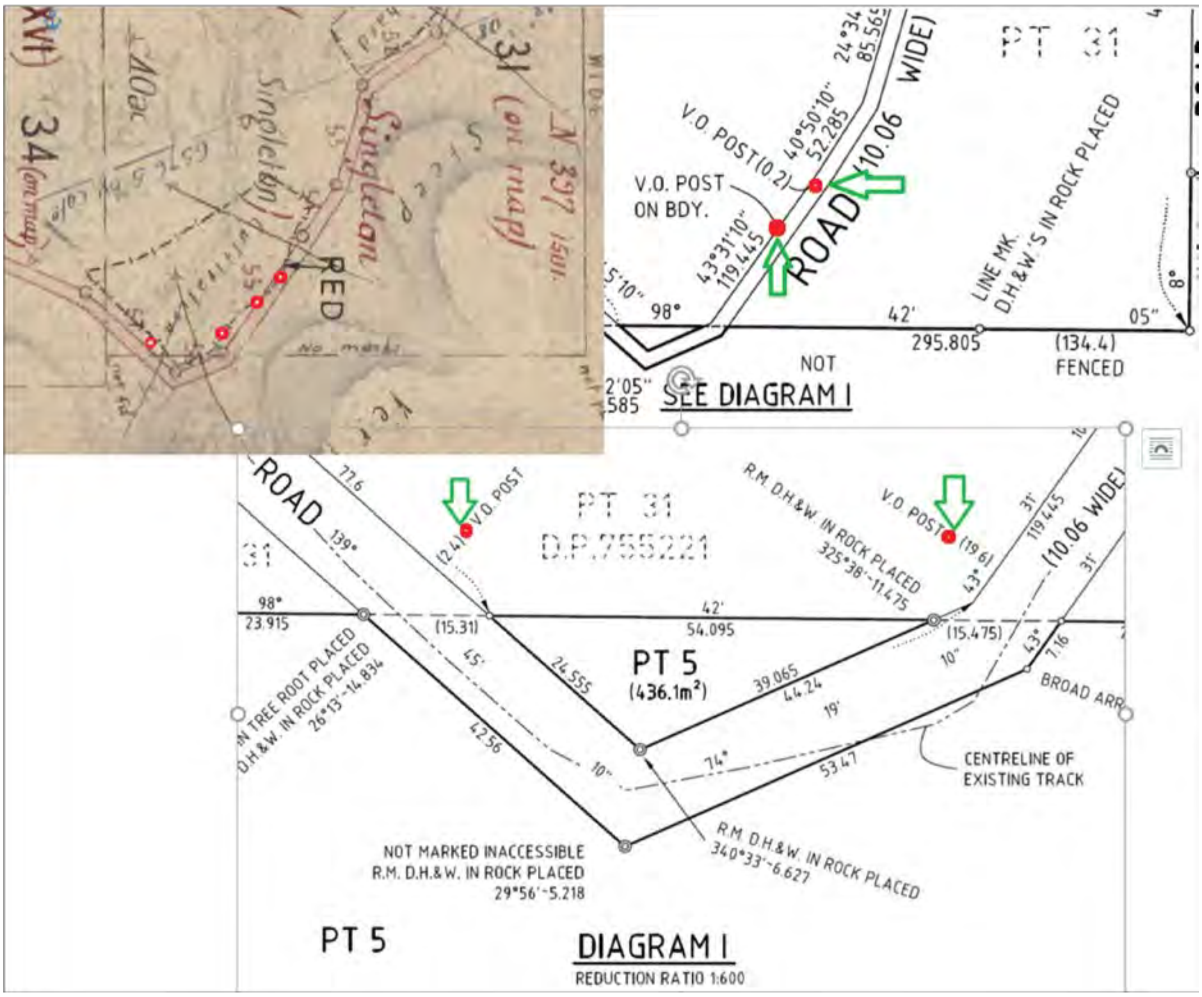


Figure 25: Extract from R3432 and Plan of Subdivision indicating fencing.

Figure 26: Sample of very old fence posts.

Another item of note is the old hut as depicted upon portion 82 in R3432, an extract of which is shown in Figure 27. A retaining wall of local rock was found in this general vicinity as shown in Figure 27, and it may be that this was the location of the cottage in 1887.

Figure 27: Retaining wall of local rock, possible site of hut in 1887.

3.5 Northern Boundary – Stage 5: Ph 81, 20, 64 and 65

Having re-established the parish road with some certainty, further calculations were then completed in relation to the reference tree found at corner S (see Figure 21) and other comparisons in the portion 64-65 plan, N4072. As shown in Figure 21, an accurate comparison was established from corner N to corner S in comparison to original dimensions.

With some renewed confidence that the calculations were reliable, the line T-U was revisited to further investigate for any marks based on the new calculations. Extensive efforts were made to find marks along this line, particularly the rock mark at corner U (Figure 28), however to no avail. The investigations extended south from corner U to the northern boundary of portion 24. Again, no marks were found.

At this point, the survey approach returned to the abuttal boundary at corner R, having closed off the traverse loop at this point. The traverse pushed eastwards along the southern boundary of Ph 64 and then looped back through the cleared valley (via corner V) to corner W. Again no marks were found, and so the traverse extended around the valley down to corner X where a stump was found in the position calculated for this corner from N4072 (Ph 65) (Figure 29).

The traverse then returned to corner V and continued along the abuttal southwards to corner Y and then across the valleys to corner Z. At Z, lockspit remains were found which agreed well with the orientation and distance from the Mahogany stump at corner X (see Figure 29).

The traverse then proceeded eastward toward corner AA. Again no line marks, blazed trees or other evidence was found. At corner AA, no tree or corner mark was found. However, this corner is not far from a transmission tower within the relevant easement, and given the regrowth nature of the flora, it is quite possible that the area had been cleared during the installation of the high voltage transmission.

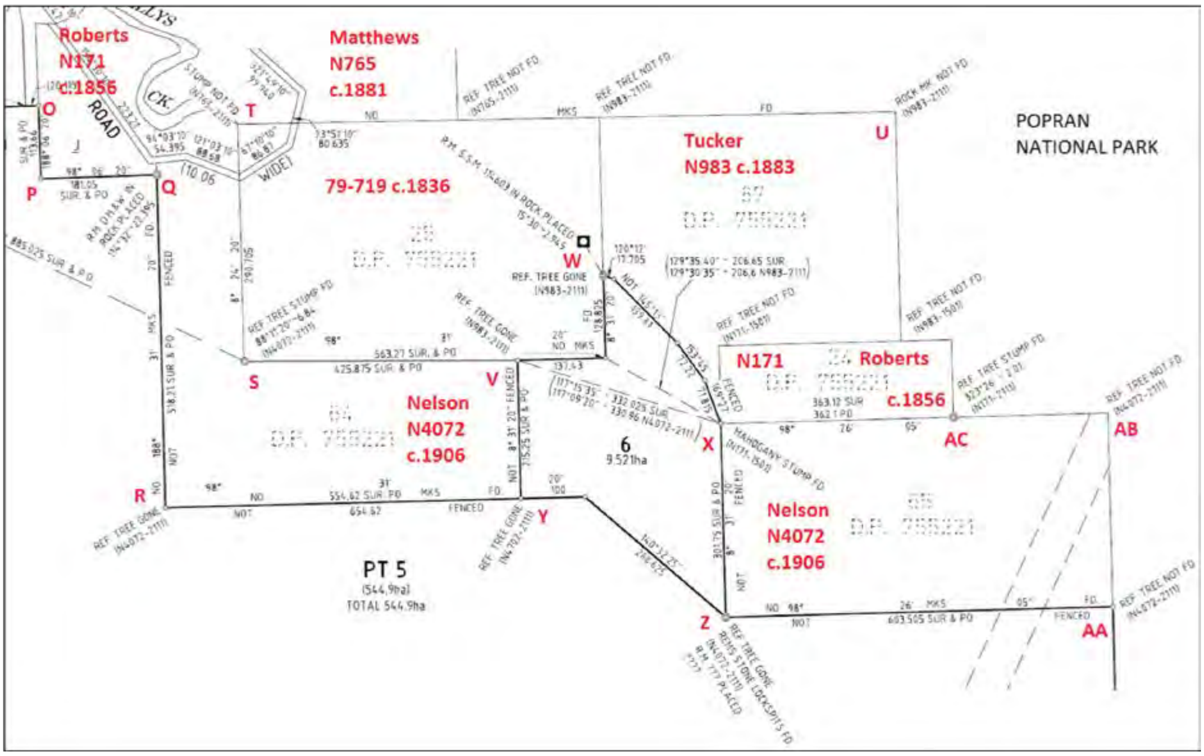


Figure 28: Extract of page 10 of the Plan of Subdivision.

Figure 29: Remains of lockspit at corner Z, Mahogany stump at corner X, and reference stump at corner AC.

At this point, it should be noted that consultation had been completed with NPWS in order to confirm the agreed definition of the common boundary extending south from corner AA. This boundary was to be a prolongation of line AA-AB southwards to lot 1. Consequently, it was necessary to investigate the definition of corner AB.

As can be seen in Figure 28, the corner AB plots within the alignment of the transmission easement. This easement, as noted earlier, is occupied and maintained regularly to protect critical infrastructure. Consequently, upon laying the corner back in, it was evident that the corner mark and tree were destroyed for the purpose of the transmission line.

The survey then investigated corner AC where the original corner was found from the Ph 24 plan (N171) by Nelson in 1906 (N4072). As noted previously, prior to the 1860s it was common for references to be noted from the corner to the mark. This corner is a case in point. Figure 29 shows the stump found and as can be seen in the Plan of Subdivision (Figure 28), the comparison was reasonable, being 1 m in excess of the N171 dimensions. Whilst the intention of N4072 was for Ph 65 to be rectangular, this was not possible based on the evidence found, and there was a 5'15" variation from square.

As can be seen from Figure 28, based on the marks found, at corners N, S, X and Z the original dimensions including bearings and distances were able to be maintained across portion 64 to the western boundary of portion 65. Further, having compared the connections between these portions, on both plans N4072 and N983, the comparisons from the Mahogany stump at corner X back to portions 25 and 64 are reasonable.

It should also be noted that the connection from the north-east corner of Ph 64 to the north-west corner of Ph 65 on N4072 is a calculation shown on the plan in red which appears to have been added after the survey was lodged with the Surveyor General. Whereas the connections shown in N983 (Ph 87) from this Mahogany stump at X (north-west corner Ph 65) to the south-east corner of portion 25 forms part of the survey information in the short line table. Consequently, this appears to be somewhat more reliable to represent actual surveyed lines rather than calculations. Indeed, it is this calculation to which the survey marks bear a reliable comparison, as the Plan of Subdivision extract in Figure 28 shows a difference of only 5' in bearing and 0.05 m in length.

Line X-AC shown in Figure 28, from the Mahogany stump to the reference gum tree, was used as the orientation for the southern boundary of Ph 65, being the survey abuttal. The portion is no longer square, but 5'15" askew as noted earlier. As a consequence, the common boundary with Popran National Park was kept parallel to the western boundary of Ph 65, consistent with the intention outlined earlier.

Having resolved the position of the majority of the boundaries of portions 64 and 65, comparisons were then made in relation to the position of the parish road as re-established versus the position as shown in N4072 (Ph 64 & 65). Figure 30 shows the results of these comparisons in yellow. On the western side, near corner Q, calculations between the road as redefined and the PO boundary indicate a minor excess of 0.255 m north-south and 0.275 m east-west. South of corner T, within diagram K, the results indicate a slight shortage north-south of 0.265 m and a discrepancy east-west of 0.45 m.

It would be speculation to attribute these differences to any one factor. However, these are considered to be quite reasonable comparisons, taking into account the evolution of the surveys, method of measurement, instrumentation and geographical factors.

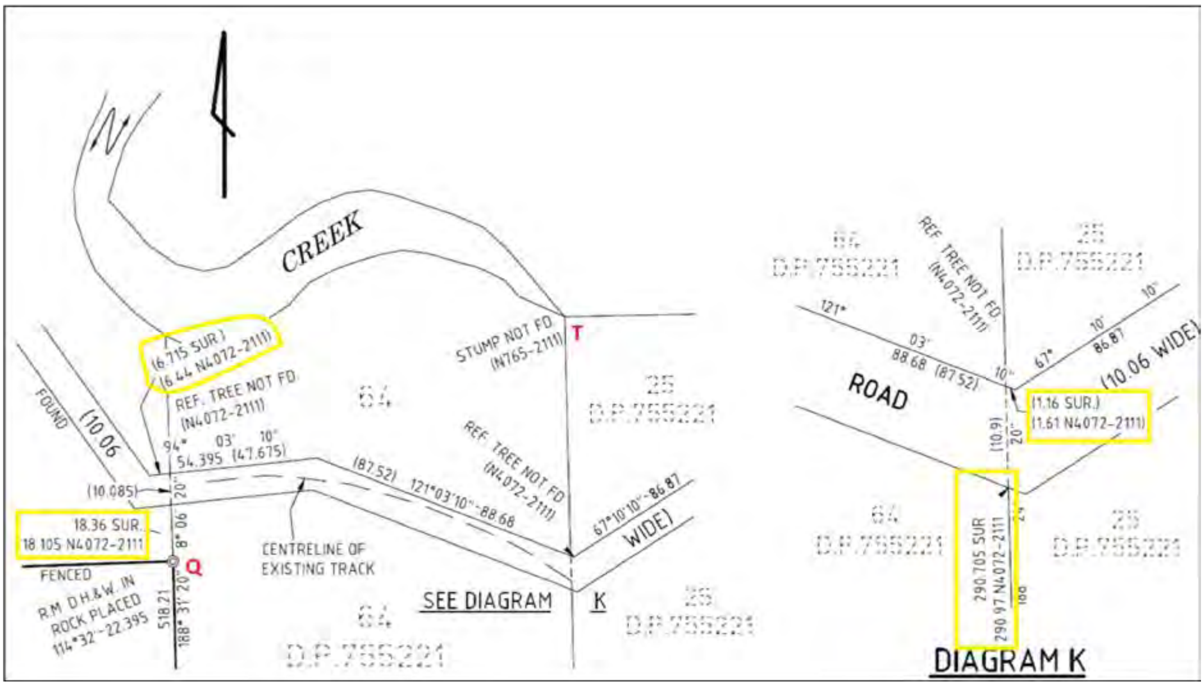


Figure 30: Extract of diagram J and K on page 10 of the Plan of Subdivision.

3.6 Northern Boundary – Stage 6: Ph 31 and 34

Having resolved the definition of the remaining northern boundary abuttals and investigated all avenues for remaining survey infrastructure in this area, focus was returned to the final definition of portions 31 & 34 shown in Figure 17. These parcels had been surveyed at the same time in October 1862 by Surveyor Commins. However, there was no survey infrastructure remaining from either plan to confirm the original boundary position along the abuttal.

Survey investigations extended northwards to the Popran Creek natural boundary in the hope that the original marks may still remain. However, no marks were found on either of the three creek boundaries. It should be noted that this is generally flat swampy marshland north of the parish road. Indeed, both plans describe the land as "large flats covered with rushes and swamp oak". All the reference trees on the plans were swamp oaks which have a relatively short life span of less than 50 years. Combining this with a high moisture content and regular flooding may contribute to the lack of any evidence remaining today.

This left a situation where, having exhausted all options for remaining survey infrastructure, the nearest marks remaining were at corner C on portion 83 and corner L on portion 82 (Figure 31), combined with what is shown to be a reliable redefinition of the parish road from R3432 through these portions.

As with all the portions, lot closes were always done in order to verify the reliability of the plan. In the case of both of these lots, miscloses were found. These were calculated as Ph 31 to be 6º for 19.5 lks (or 3.9 m) and Ph 34 to be 4º 7 lks (or 1.41 m). As these miscloses were in roughly the same direction, the accumulated misclose for Mr Commins' survey was 5º 40' for 26.5 lks (or 5.33 m).

Further, there is also some doubt as to the traverse connections shown within the two plans. In Figure 31, the common northern corner of the two portions is circled in blue. This corner is referred to as corner A in both of the plans. In Figure 32, the same corner is also circled blue for reference.

In the case of N397, a traverse extended westward in order to connect to the mouth of Popran Creek with a series of lines extending from corner A as shown in Figure 31. Again in N453, the traverse extended westwards in order to complete the traverse across the water frontage of Popran Creek.

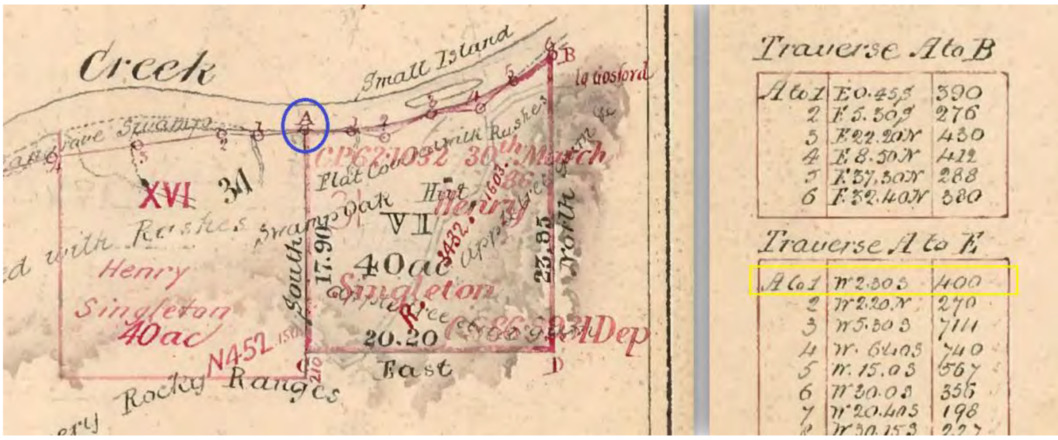


Figure 31: Extract from N397 (Ph 31) and traverse notes.

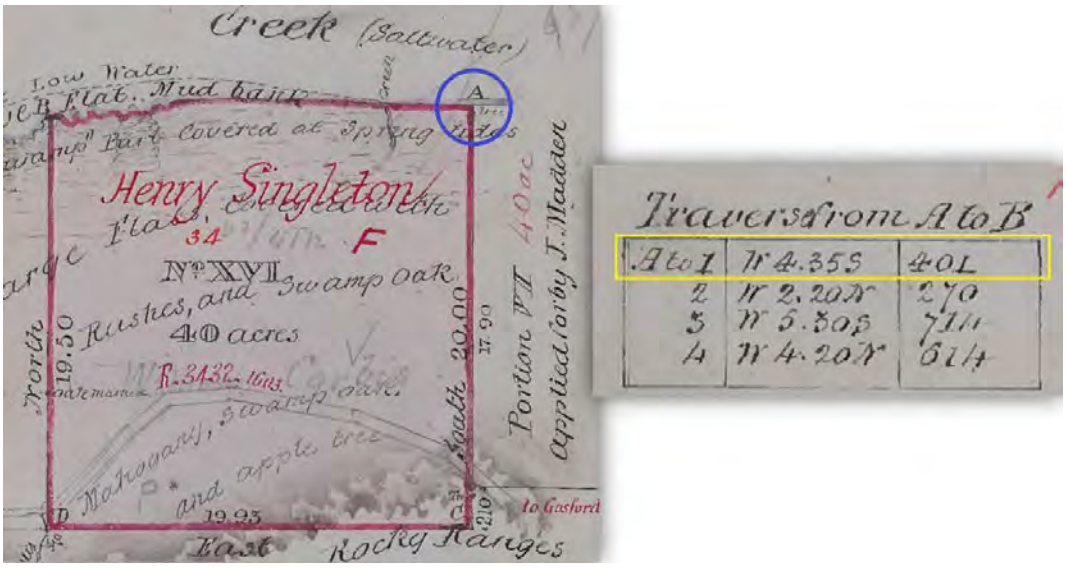


Figure 32: Extract from N453 (Ph 34) and traverse notes.

Interestingly, the first connection from corner A in each plan is different, whereas the second and third lines in each traverse are identical. It seems unusual that a traverse would have been exactly identical and parallel to the first survey. Why would the same surveyor not utilise the same traverse line when doing the survey of these parcels at the same time?

By calculating the misclose between these two lines, we come up with a line bearing due north for 14.6 lks. This compares very closely with the reference tree connection at corner A, being shown from the corner to the tree as south 15 lks. These differences seem to suggest

that the calculations may have been from the reference tree itself rather than the corner. Ultimately, these results create doubts as to the reliability of the surveys.

Consequently, the definition of the portion 31 & 34 boundaries relied in the main on the definition of the adjoining portions. Referring to Figure 33, corner H was defined from using the position of the corner shown in the parish road survey, R3432. This proved a reliable comparison in distance to the original mark found at corner C.

The southern boundary of portion 34 was ultimately determined from the last survey on record of this location, being the road plan R3432. An extract of this section of the plan shown in Figure 34 indicates that Surveyor Finn had found both the intersection of the road boundary and portion 83 (as per Tucker in 1883) as well as the north-west corner of Ph 34 (as per Commins in 1862). The comparison of azimuth between R3432 and this orientation results in a bearing for the southern boundary of Ph 34 being 99º10’10”.

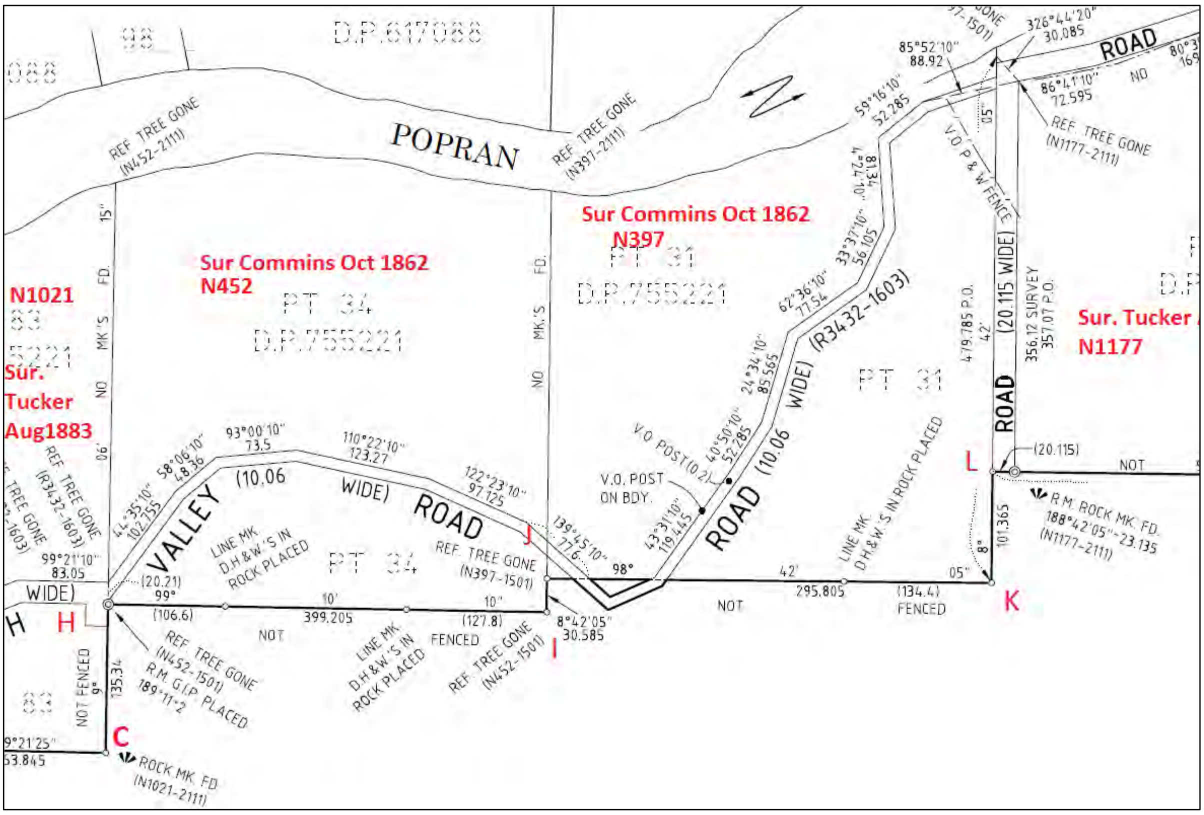


Figure 33: Extract from page 9 of the Plan of Subdivision.

Turning to the eastern boundary of Ph 31, the north-east corner of Ph 82 was re-established from the connections provided in the parish road plan R3432. Using the reference mark broad arrow found at L (see Figure 33), the boundary of Ph 31 was re-established at PO length. A right angle was then maintained at the south-east corner of Ph 31, such that the bearings of lines L-K-J-I are square as shown in the Ph 31 plan (397-1501).

The length of the southern boundaries of Ph 31 & 34 combined to be a total of 807.29 m. However, based on the refixed position of corners H and K, the survey length is 803.79 m, leaving a shortage of 3.5 m. Consequently, in the absence of any evidence for the position of the common north-south boundary between Ph 31 and Ph 34, the shortage was proportioned

between these two portions, resulting in the final dimensions and position of line I-J as shown in the Plan of Subdivision.

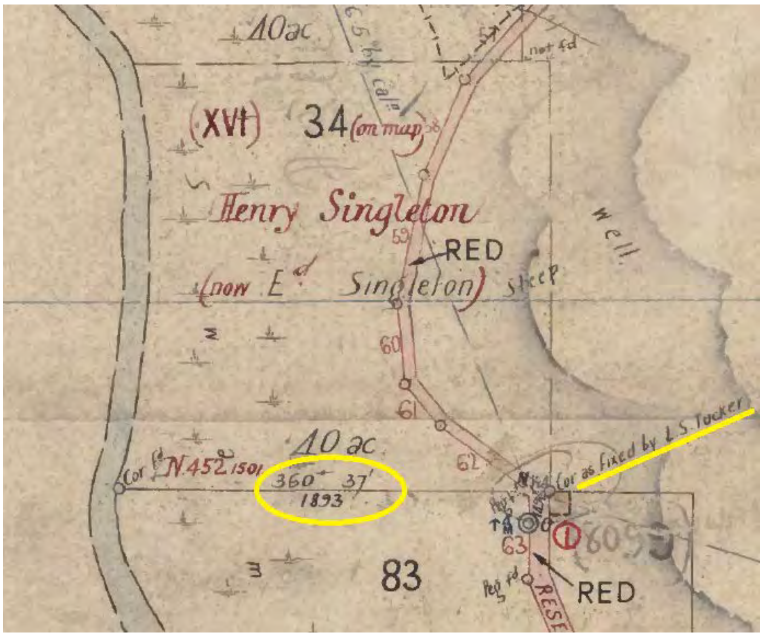


Figure 34: Extract from R3432 showing the connection to Ph 83 and 34 marks.

The resultant length of line I-J, being 30.585 m, was 11.66 m shorter than PO. Whilst it is disappointing to disagree with a previous survey by such a large amount, the passage of time, loss of infrastructure and method of measurement all contribute to these circumstances. Further, the issues encountered with the misclose within these portions also create doubt as to the reliability of the original survey. Ultimately, these differences are not only reflected in this survey, but also in the earlier survey of the parish road.

4 CONCLUDING REMARKS

This paper has demonstrated some of the challenges that surveyors face in redefining boundaries. The colloquialism that ‘surveying is an art not a science’ is reflective of the challenges, interpretations and judgments that have been made in undertaking this survey. The project has been a schooling of the project team in the historical methods of surveying, combined with the means to interpret them in the modern landscape. The project surveyors have gathered a detailed understanding of traditional methods, resources and techniques used to complete portion surveys in a very different time.

Indeed, on many occasions it has been reinforced that it is our role to imagine and place ourselves in the mindset of the original surveyor, taking into account their reliance on angular measurement, their need to minimise manual calculations and use the resources available to them on site.

The results of the various ages of the surveys have also been a demonstration of the value in good technique and complying with sound survey practice. Indeed, the older plans from pre-1860 have been a challenge to redefine, whereas in comparison the 1860-1910 surveys, followed by the surveys of the last century, show an incremental improvement in existence of marks, accuracy and subsequent reliability.

The contracting of such a large project has created the opportunity for young surveyors to experience such challenging definitions, whereas previously these surveys may have been completed by government survey teams. The contracting of more of these surveys may also provide opportunity for a broader sector of the profession to experience this surveying environment.

REFERENCES

Department of Environment, Climate Change & Water (2010) Due Diligence Code of Practice for the Protection of Aboriginal Objects in New South Wales, http://www.environment.nsw.gov.au/resources/cultureheritage/ddcop/10798ddcop.pdf (accessed Feb 2017).

DFSI Spatial Services (2016a) Surveyor General's Direction No. 3: Control for Cadastral Surveys, http://spatialservices.finance.nsw.gov.au/surveying/publications/surveyor_generals_directions (accessed Feb 2017).

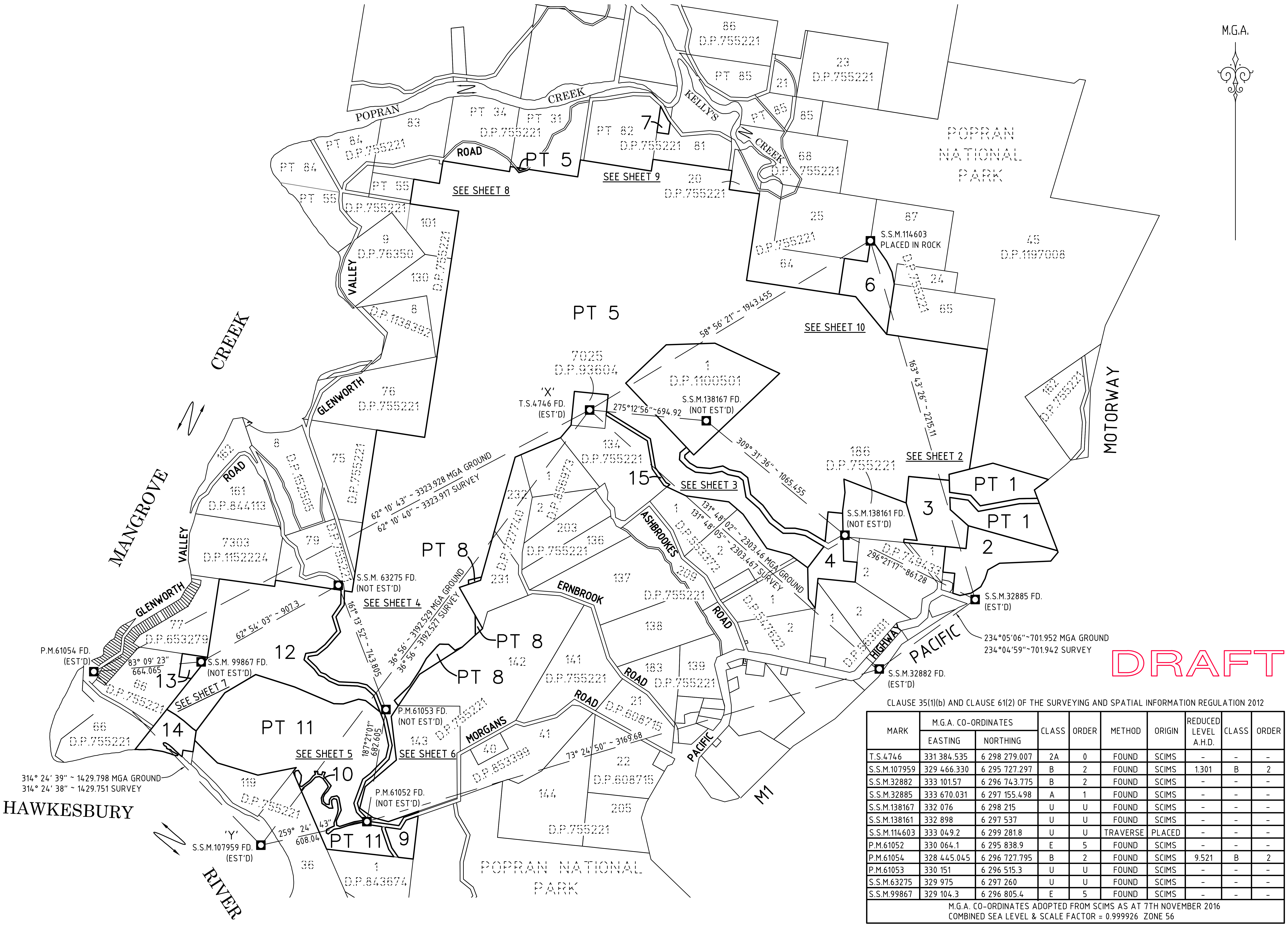
DFSI Spatial Services (2016b) Surveyor General's Direction No. 10: Surveys of Crown Land, http://spatialservices.finance.nsw.gov.au/surveying/publications/surveyor_generals_directions (accessed Feb 2017).

Marshall I.H. (2002) *Marking the landscape: A short history of survey marking in New South Wales* (2nd edition), Land and Property Information, Bathurst, 128pp.

APPENDIX

The following 12 pages of the appendix contain the Plan of Subdivision of:

- Lot 161 in DP 755221.
- Lot 7020 in DP 1065007.
- Lot 7021 in DP 1114447.
- Lot 7310 in DP 1155132.
- Lot 7040 in DP 1116103.
- Lot 7027 in DP 1051931.
- Lot 7041 and 7042 in DP 1116109.
- Lot 2597 in DP 1205726.



CLAUSE 35(1)(b) AND CLAUSE 61(2) OF THE SURVEYING AND SPATIAL INFORMATION REGULATION 2012

MARK	M.G.A. CO-ORDINATES		CLASS	ORDER	METHOD	ORIGIN	REDUCED LEVEL A.H.D.	CLASS	ORDER
	EASTING	NORTHING							
T.S.4746	331 384.535	6 298 279.007	2A	0	FOUND	SCIMS	-	-	-
S.S.M.107959	329 466.330	6 295 727.297	B	2	FOUND	SCIMS	1.301	B	2
S.S.M.32882	333 101.57	6 296 743.775	B	2	FOUND	SCIMS	-	-	-
S.S.M.32885	333 670.031	6 297 155.498	A	1	FOUND	SCIMS	-	-	-
S.S.M.138167	332 076	6 298 215	U	U	FOUND	SCIMS	-	-	-
S.S.M.138161	332 898	6 297 537	U	U	FOUND	SCIMS	-	-	-
S.S.M.114603	333 049.2	6 299 281.8	U	U	TRAVERSE	PLACED	-	-	-
P.M.61052	330 064.1	6 295 838.9	E	5	FOUND	SCIMS	-	-	-
P.M.61054	328 445.045	6 296 727.795	B	2	FOUND	SCIMS	9.521	B	2
P.M.61053	330 151	6 296 515.3	U	U	FOUND	SCIMS	-	-	-
S.S.M.63275	329 975	6 297 260	U	U	FOUND	SCIMS	-	-	-
S.S.M.99867	329 104.3	6 296 805.4	E	5	FOUND	SCIMS	-	-	-

M.G.A. CO-ORDINATES ADOPTED FROM SCIMS AS AT 7TH NOVEMBER 2016
COMBINED SEA LEVEL & SCALE FACTOR = 0.999926 ZONE 56

Surveyor: ANTHONY JAMES OLIVER
Date of Survey:
Surveyor's Ref: 190299S-DP-001-B
2016M7100(428) PARTIAL SURVEY
2016M7100(1746) ADDITIONAL SHEETS

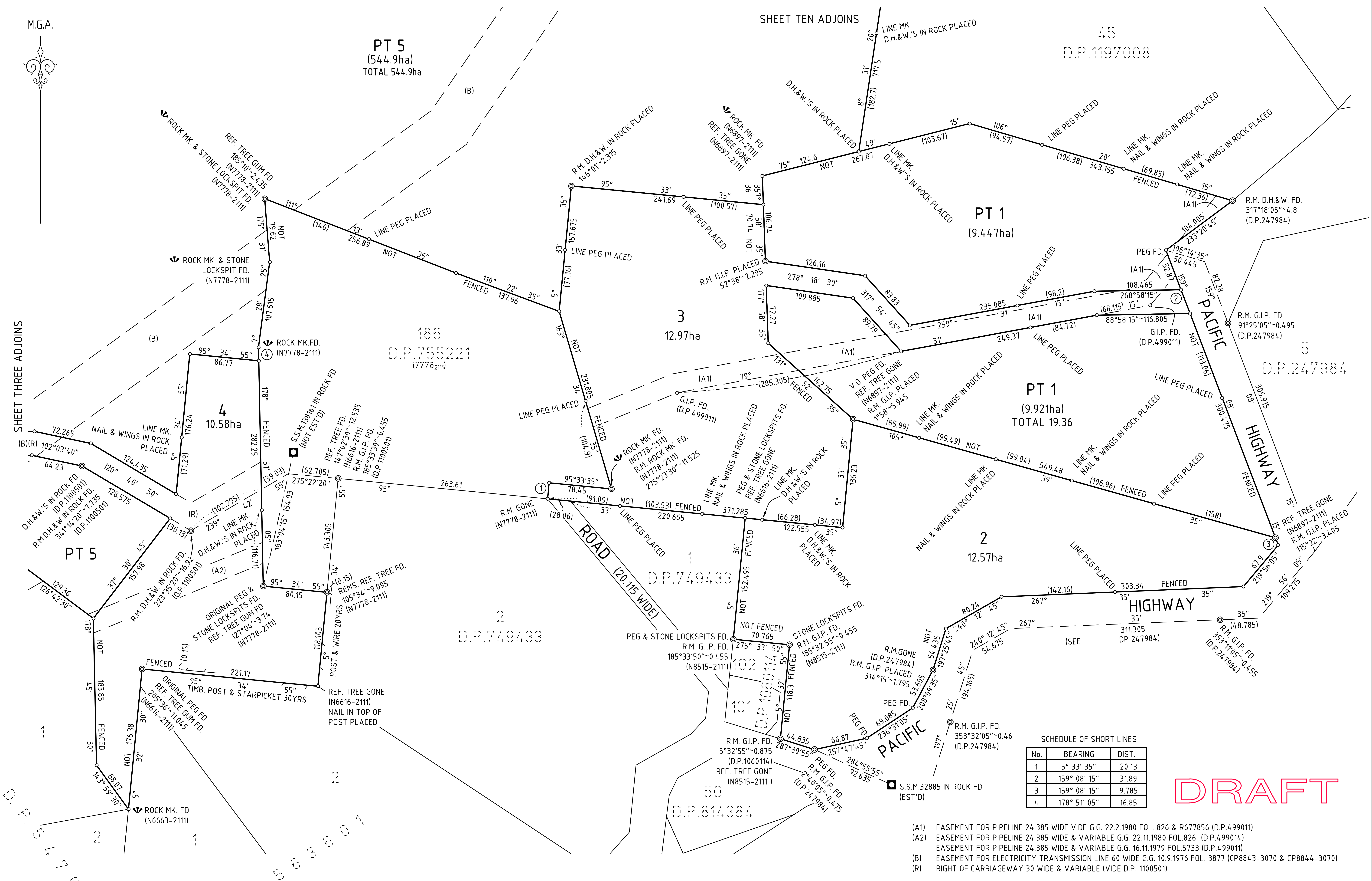
PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.

LGA: CENTRAL COAST
Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK
Subdivision No: N/A
Lengths are in metres. Reduction Ratio: 1:15000

Registered

D.P.

10 | 20 | 30 | 40 | 50 | Table of mm | 90 | 100 | 110 | 120 | 130 | 140



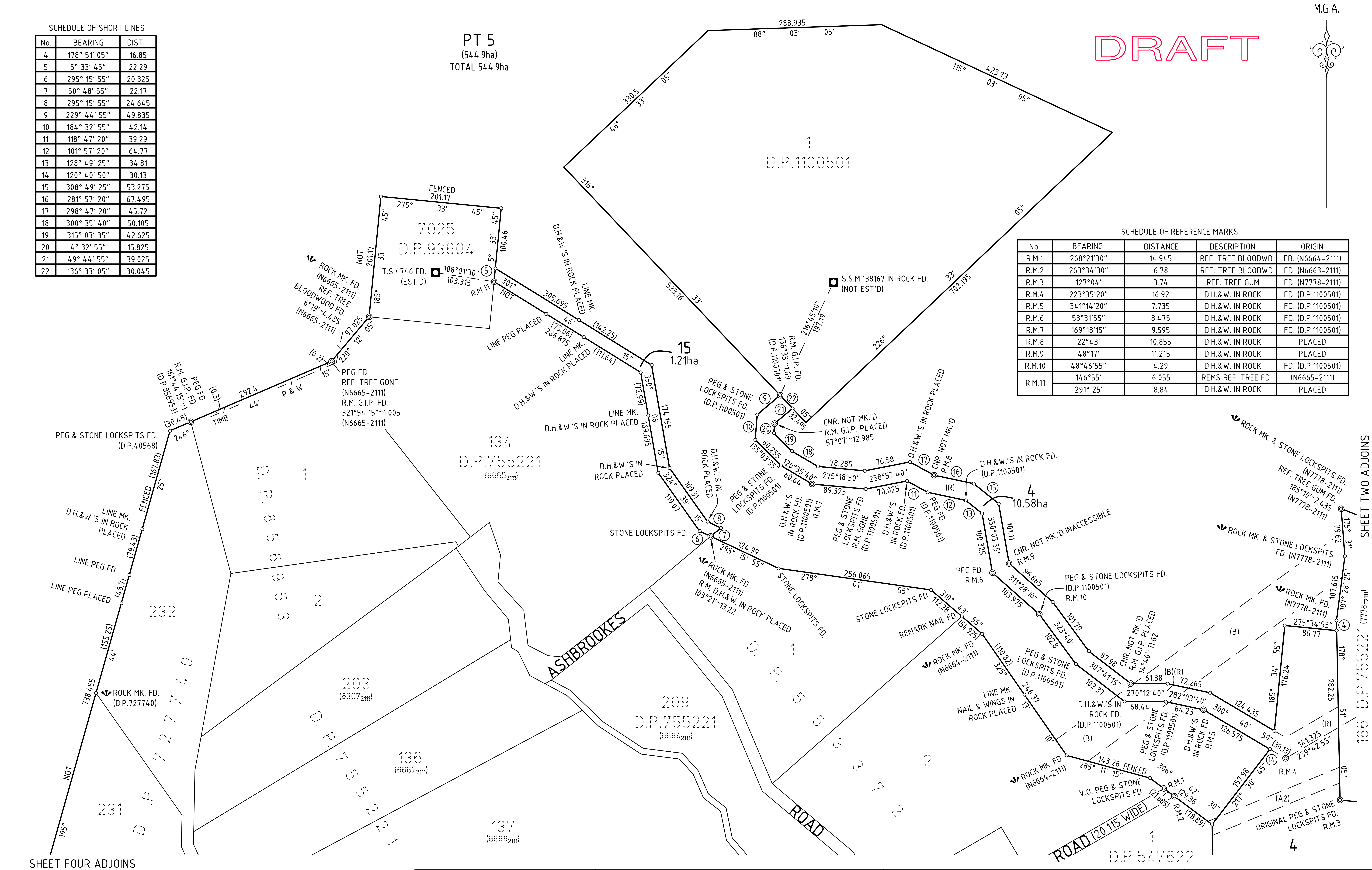
SCHEDULE OF SHORT LINES

No.	BEARING	DIST.
1	5° 33' 35"	20.13
2	159° 08' 15"	31.89
3	159° 08' 15"	9.785
4	178° 51' 05"	16.85

(A1) EASEMENT FOR PIPELINE 24.385 WIDE VIDE G.G. 22.2.1980 FOL. 826 & R677856 (D.P.499011)
(A2) EASEMENT FOR PIPELINE 24.385 WIDE & VARIABLE G.G. 22.11.1980 FOL.826 (D.P.499014)
EASEMENT FOR PIPELINE 24.385 WIDE & VARIABLE G.G. 16.11.1979 FOL.5733 (D.P.499011)
(B) EASEMENT FOR ELECTRICITY TRANSMISSION LINE 60 WIDE G.G. 10.9.1976 FOL. 3877 (CP8843-3070 & CP8844-3070)
(R) RIGHT OF CARRIAGEWAY 30 WIDE & VARIABLE (VIDE D.P. 1100501)

Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:3000	Registered	D.P.

10 20 30 40 50 Table of mm 90 100 110 120 130 140



SCHEDULE OF SHORT LINES

No.	BEARING	DIST.
4	178° 51′ 05″	16.85
5	5° 33′ 45″	22.29
6	295° 15′ 55″	20.325
7	50° 48′ 55″	22.17
8	295° 15′ 55″	24.645
9	229° 44′ 55″	49.835
10	184° 32′ 55″	42.14
11	118° 47′ 20″	39.29
12	101° 57′ 20″	64.77
13	128° 49′ 25″	34.81
14	120° 40′ 50″	30.13
15	308° 49′ 25″	53.275
16	281° 57′ 20″	67.495
17	298° 47′ 20″	45.72
18	300° 35′ 40″	50.105
19	315° 03′ 35″	42.625
20	4° 32′ 55″	15.825
21	49° 44′ 55″	39.025
22	136° 33′ 05″	30.045

SCHEDULE OF REFERENCE MARKS

No.	BEARING	DISTANCE	DESCRIPTION	ORIGIN
R.M.1	268°21′30″	14.945	REF. TREE BLOODWD	FD. (N6664-2111)
R.M.2	263°34′30″	6.78	REF. TREE BLOODWD	FD. (N6663-2111)
R.M.3	127°04′	3.74	REF. TREE GUM	FD. (N7778-2111)
R.M.4	223°35′20″	16.92	D.H.&W. IN ROCK	FD. (D.P.1100501)
R.M.5	341°14′20″	7.735	D.H.&W. IN ROCK	FD. (D.P.1100501)
R.M.6	53°31′55″	8.475	D.H.&W. IN ROCK	FD. (D.P.1100501)
R.M.7	169°18′15″	9.595	D.H.&W. IN ROCK	FD. (D.P.1100501)
R.M.8	22°43′	10.855	D.H.&W. IN ROCK	PLACED
R.M.9	48°17′	11.215	D.H.&W. IN ROCK	PLACED
R.M.10	48°46′55″	4.29	D.H.&W. IN ROCK	FD. (D.P.1100501)
R.M.11	146°55′	6.055	REMS REF. TREE FD.	(N6665-2111)
	291° 25′	8.84	D.H.&W. IN ROCK	PLACED

(A2) EASEMENT FOR PIPELINE 24.385 WIDE & VARIABLE G.G. 22.11.1980 FOL.826 (D.P.499014)
EASEMENT FOR PIPELINE 24.385 WIDE & VARIABLE G.G. 16.11.1979 FOL.5733 (D.P.499011)
(B) EASEMENT FOR ELECTRICITY TRANSMISSION LINE 60 WIDE G.G. 10.9.1976 FOL. 3877 (CP8843-3070 & CP8844-3070)
(R) RIGHT OF CARRIAGEWAY 30 WIDE & VARIABLE (VIDE D.P. 1100501)

Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:4000	Registered	D.P.

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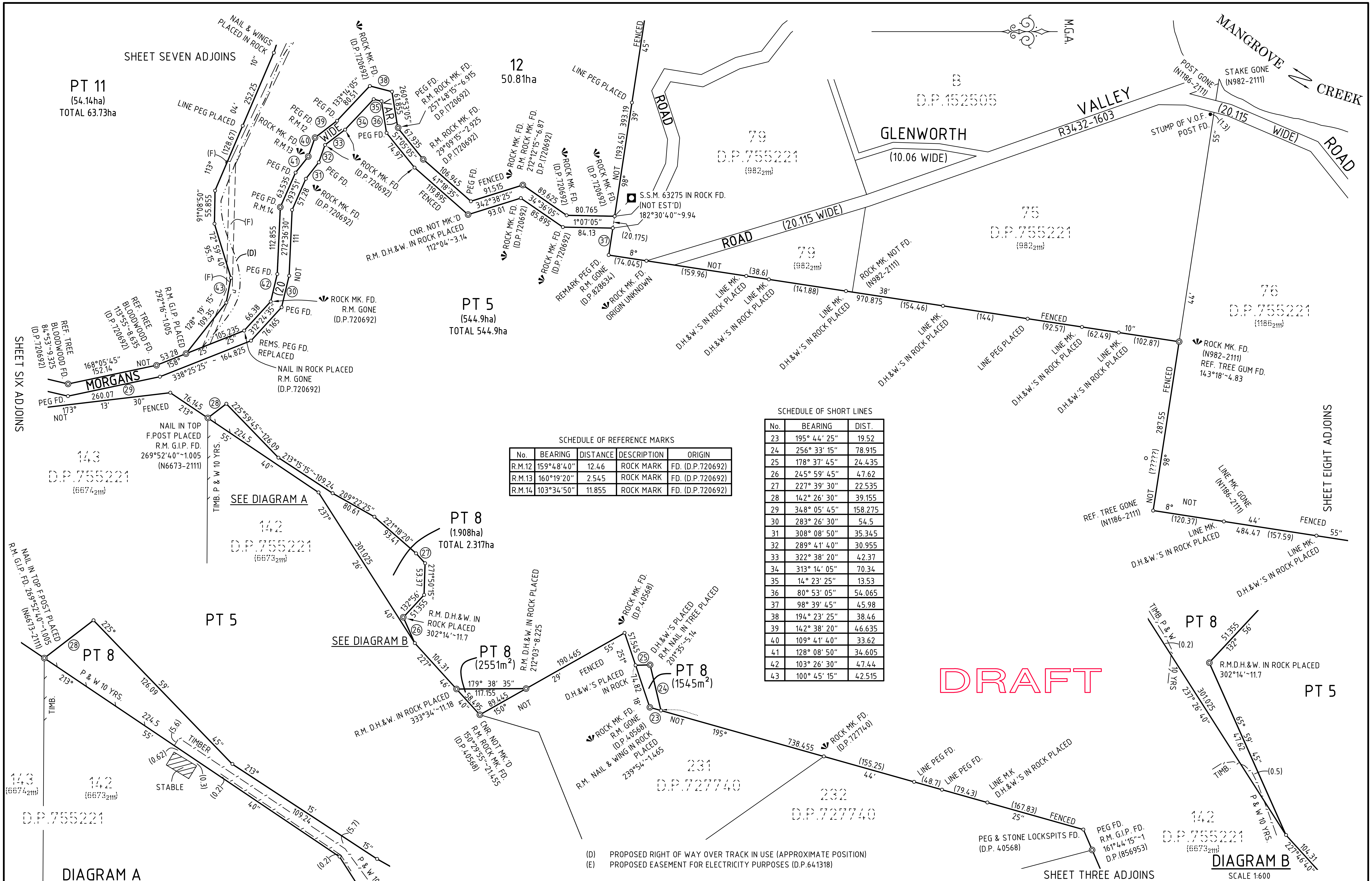


SCHEDULE OF REFERENCE MARKS

No.	BEARING	DISTANCE	DESCRIPTION	ORIGIN
R.M.12	159°48'40"	12.46	ROCK MARK	FD. (D.P.720692)
R.M.13	160°19'20"	2.545	ROCK MARK	FD. (D.P.720692)
R.M.14	103°34'50"	11.855	ROCK MARK	FD. (D.P.720692)

SCHEDULE OF SHORT LINES

No.	BEARING	DIST.
23	195° 44' 25"	19.52
24	256° 33' 15"	78.915
25	178° 37' 45"	24.435
26	245° 59' 45"	47.62
27	227° 39' 30"	22.535
28	142° 26' 30"	39.155
29	348° 05' 45"	158.275
30	283° 26' 30"	54.5
31	308° 08' 50"	35.345
32	289° 41' 40"	30.955
33	322° 38' 20"	42.37
34	313° 14' 05"	70.34
35	14° 23' 25"	13.53
36	80° 53' 05"	54.065
37	98° 39' 45"	45.98
38	194° 23' 25"	38.46
39	142° 38' 20"	46.635
40	109° 41' 40"	33.62
41	128° 08' 50"	34.605
42	103° 26' 30"	47.44
43	100° 45' 15"	42.515

(D) PROPOSED RIGHT OF WAY OVER TRACK IN USE (APPROXIMATE POSITION)
(E) PROPOSED EASEMENT FOR ELECTRICITY PURPOSES (D.P.641318)

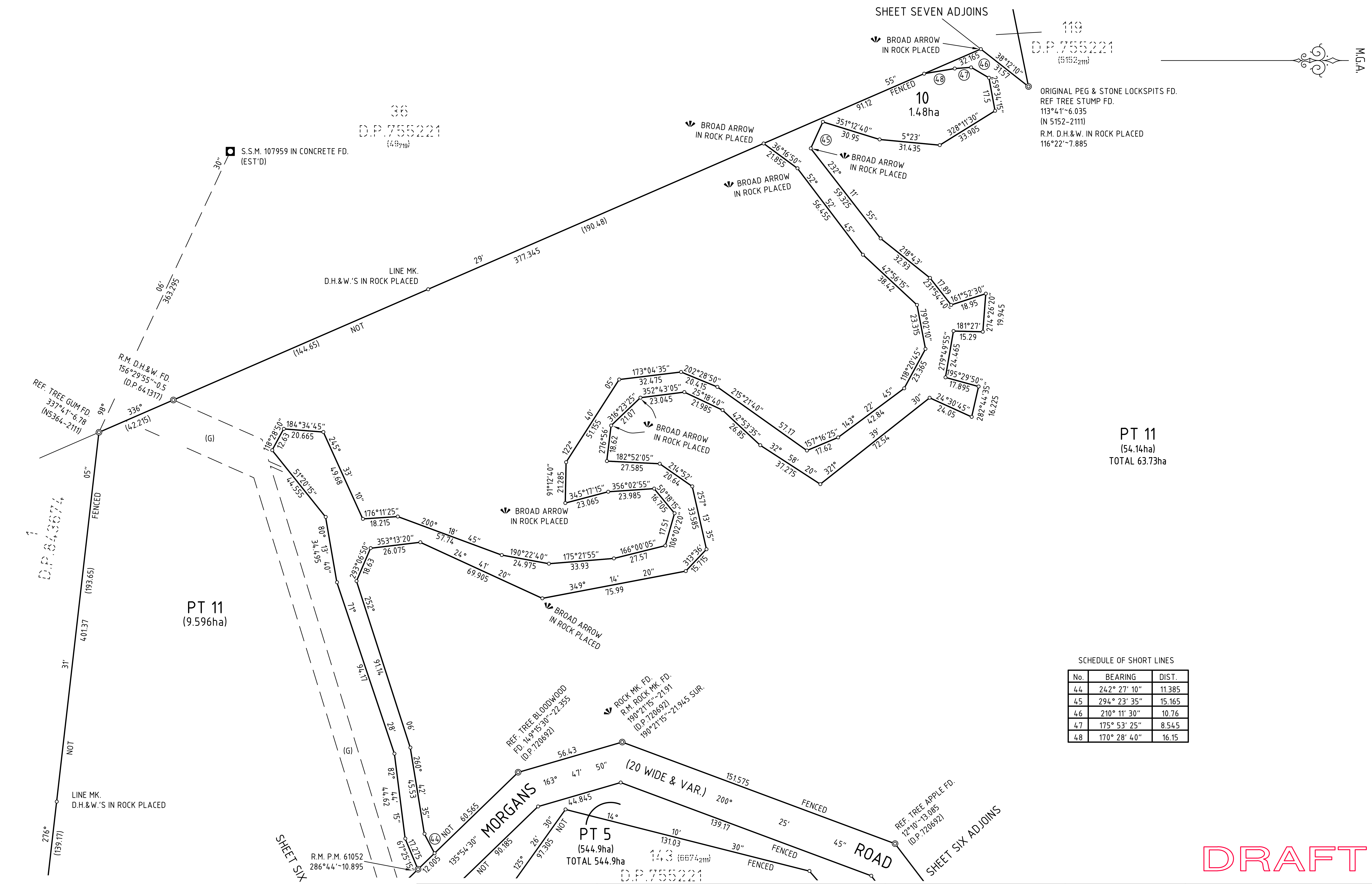
Surveyor: ANTHONY JAMES OLIVER
Date of Survey:
Surveyor's Ref: 190299S-DP-001-B
2016M7100(428) PARTIAL SURVEY
2016M7100(1746) ADDITIONAL SHEETS

PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.

LGA: CENTRAL COAST
Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK
Subdivision No: N/A
Lengths are in metres. Reduction Ratio: 1:4000

Registered

D.P.

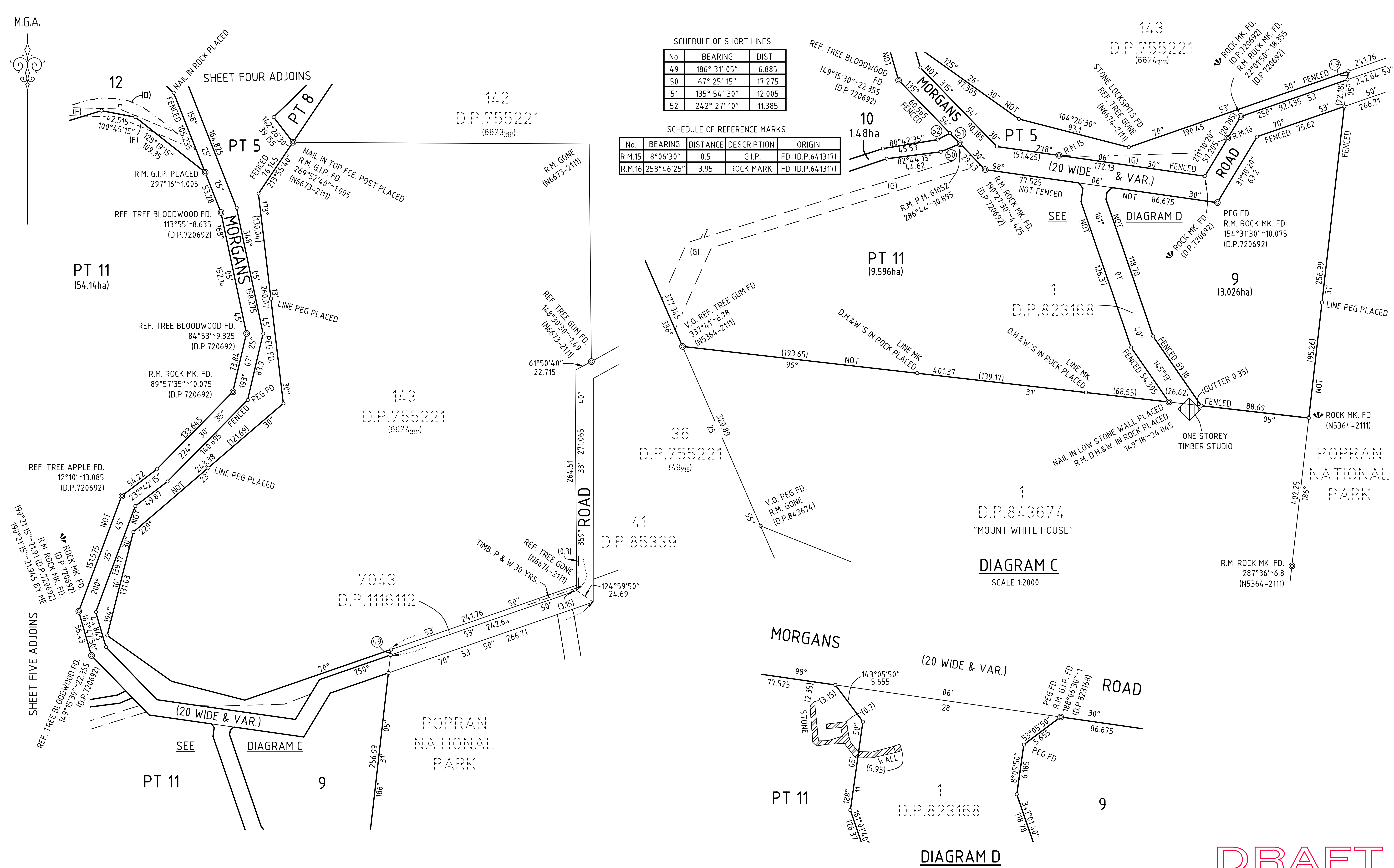


SCHEDULE OF SHORT LINES

No.	BEARING	DIST.
44	242° 27' 10"	11.385
45	294° 23' 35"	15.165
46	210° 11' 30"	10.76
47	175° 53' 25"	8.545
48	170° 28' 40"	16.15

(G) PROPOSED EASEMENT FOR ELECTRICITY PURPOSES (D.P. 641317)

Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:2000	Registered	D.P.



SCHEDULE OF SHORT LINES

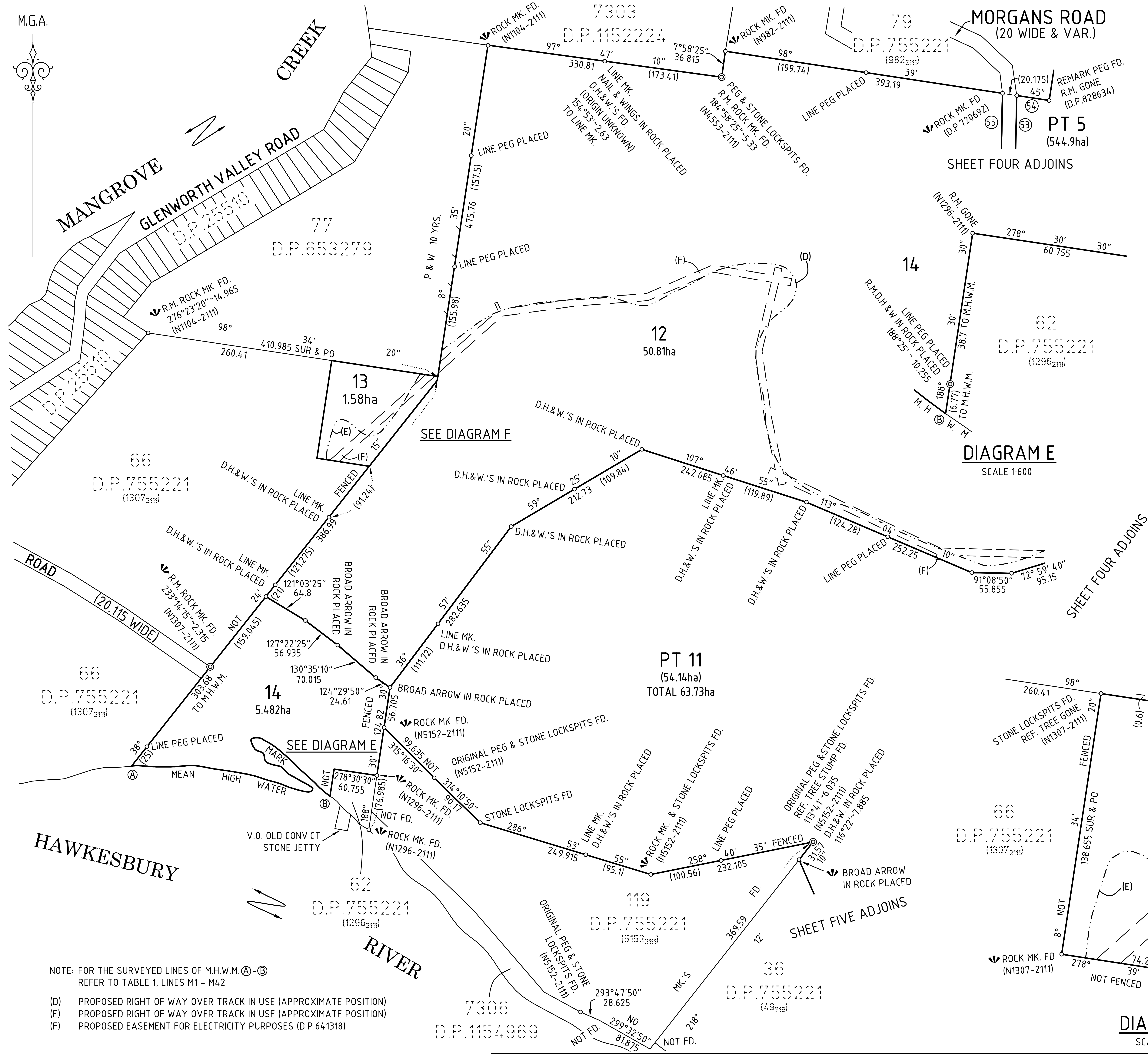
No.	BEARING	DIST.
49	186° 31' 05"	6.885
50	67° 25' 15"	17.275
51	135° 54' 30"	12.005
52	242° 27' 10"	11.385

SCHEDULE OF REFERENCE MARKS

No.	BEARING	DISTANCE	DESCRIPTION	ORIGIN
R.M.15	8°06'30"	0.5	G.I.P.	FD. (D.P.641317)
R.M.16	258°46'25"	3.95	ROCK MARK	FD. (D.P.641317)

(D) PROPOSED RIGHT OF WAY OVER TRACK IN USE (APPROXIMATE POSITION)
(F) PROPOSED EASEMENT FOR ELECTRICITY PURPOSES (D.P.641318)
(G) PROPOSED EASEMENT FOR ELECTRICITY PURPOSES (D.P. 641317)

Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:3000	Registered	D.P.



NOTE: FOR THE SURVEYED LINES OF M.H.W.M. Ⓐ-Ⓑ REFER TO TABLE 1, LINES M1 – M42

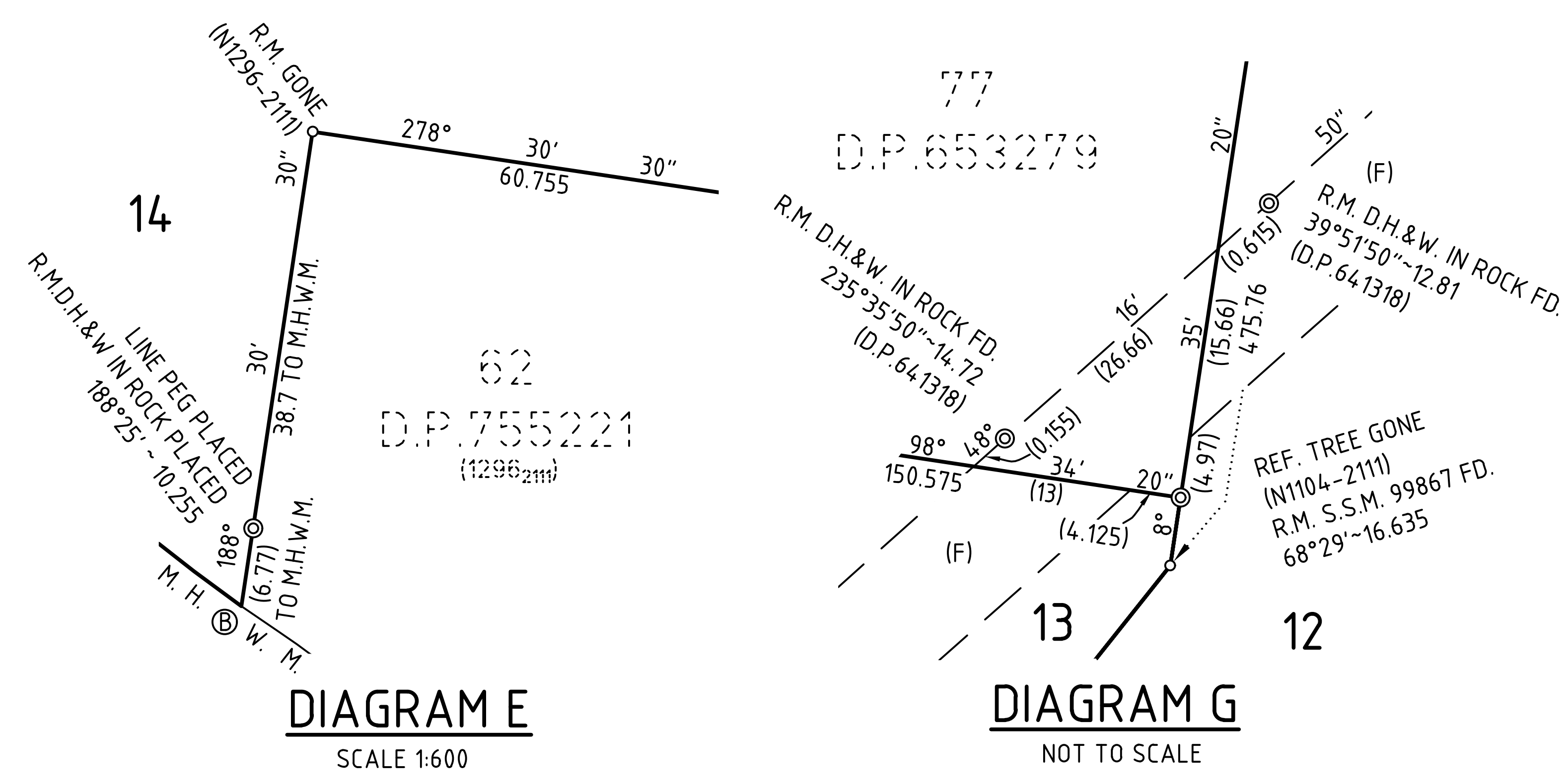
(D) PROPOSED RIGHT OF WAY OVER TRACK IN USE (APPROXIMATE POSITION)
(E) PROPOSED RIGHT OF WAY OVER TRACK IN USE (APPROXIMATE POSITION)
(F) PROPOSED EASEMENT FOR ELECTRICITY PURPOSES (D.P.641318)

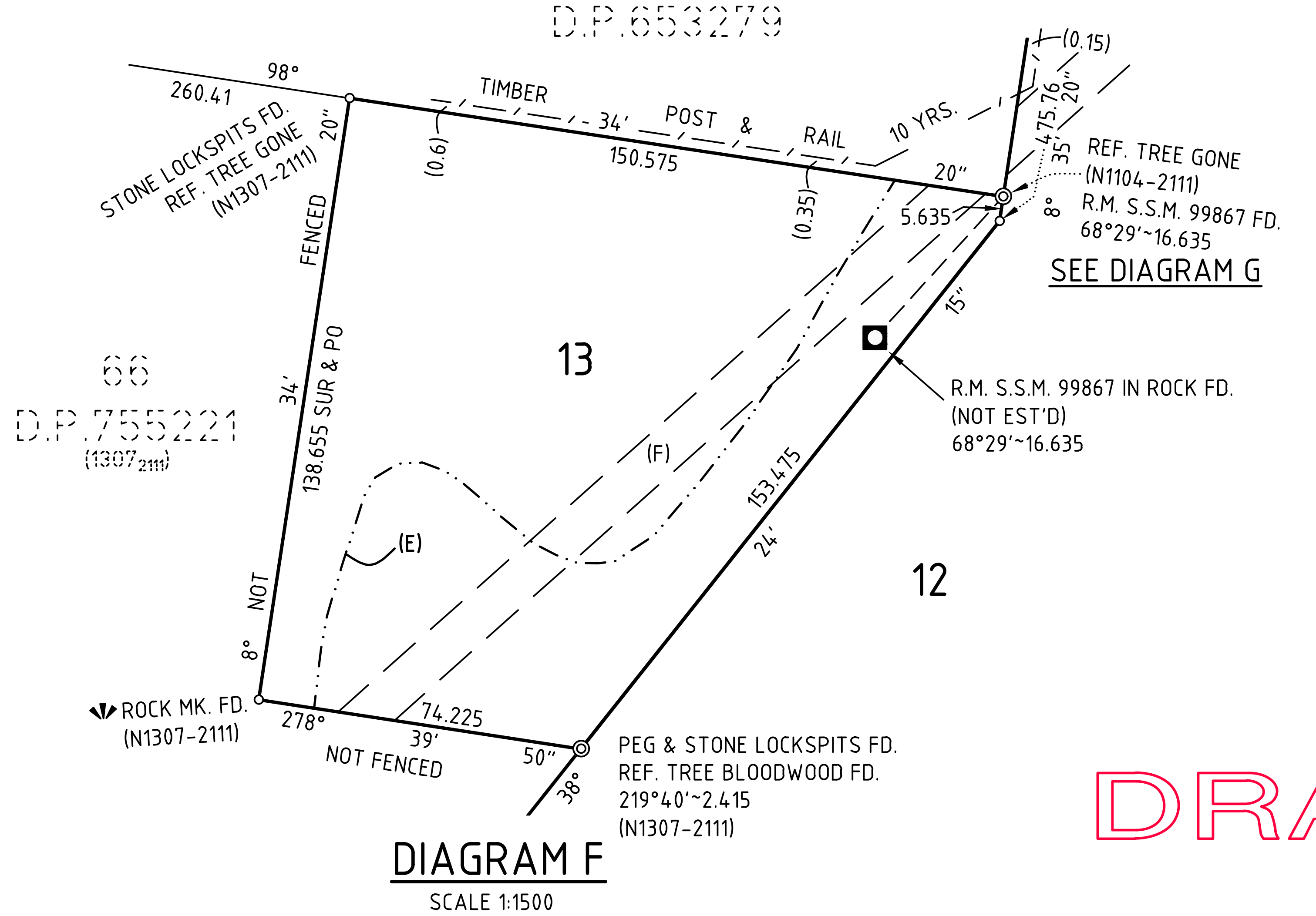
SCHEDULE OF SHORT LINES

No.	BEARING	DIST.
53	1° 07′ 05″	84.135
54	98° 39′ 45″	45.98
55	181° 07′ 05″	80.765

SCHEDULE OF M.H.W.M. A–B BOUNDARIES

	No.	BEARING	CHORD
STARTING AT 'A'	M1	258° 56′ 30″	5.25
	M2	274° 50′ 35″	24.07
	M3	267° 28′ 40″	11.775
	M4	269° 40′ 20″	22.73
	M5	263° 15′ 20″	23.715
	M6	283° 23′ 35″	25.42
	M7	276° 33′ 25″	24.175
	M8	280° 22′ 25″	21.445
	M9	280° 12′ 10″	15.755
	M10	290° 23′ 15″	21.57
	M11	281° 13′ 15″	14.92
	M12	296° 43′ 05″	12.5
	M13	285° 57′ 15″	21.3
	M14	279° 36′ 30″	10.965
	M15	243° 54′	5.725
	M16	254° 17′ 10″	2.51
	M17	157°51′	1.21
	M18	128° 51′ 55″	3.25
	M19	122° 51′ 30″	11.04
	M20	115° 24′ 05″	10.12
	M21	123° 17′ 25″	8.855
	M22	114° 24′ 40″	12.225
	M23	152° 09′ 25″	11.52
	M24	87° 45′ 15″	13.585
	M25	123° 28′ 25″	13.22
	M26	170° 00′ 55″	10.65
	M27	122° 09′	6.805
	M28	99° 02′ 25″	6.51
	M29	155° 08′ 35″	13.965
	M30	202° 20′ 45″	5.51
	M31	266° 47′ 35″	11.135
	M32	308° 15′	5.755
	M33	296° 38′ 45″	14.65
	M34	310° 04′ 25″	19.995
	M35	306° 39′ 10″	10.09
	M36	316° 45′ 25″	10.33
	M37	317° 50′ 15″	11.475
	M38	30° 52′ 10″	8.03
	M39	312° 23′ 15″	11.27
	M40	312° 15′ 35″	11.245
	M41	317° 44′ 20″	12.4
FINISHING AT 'B'	M42	306° 51′ 15″	13.545
	M1–M42	278° 37′ 10″	281.76

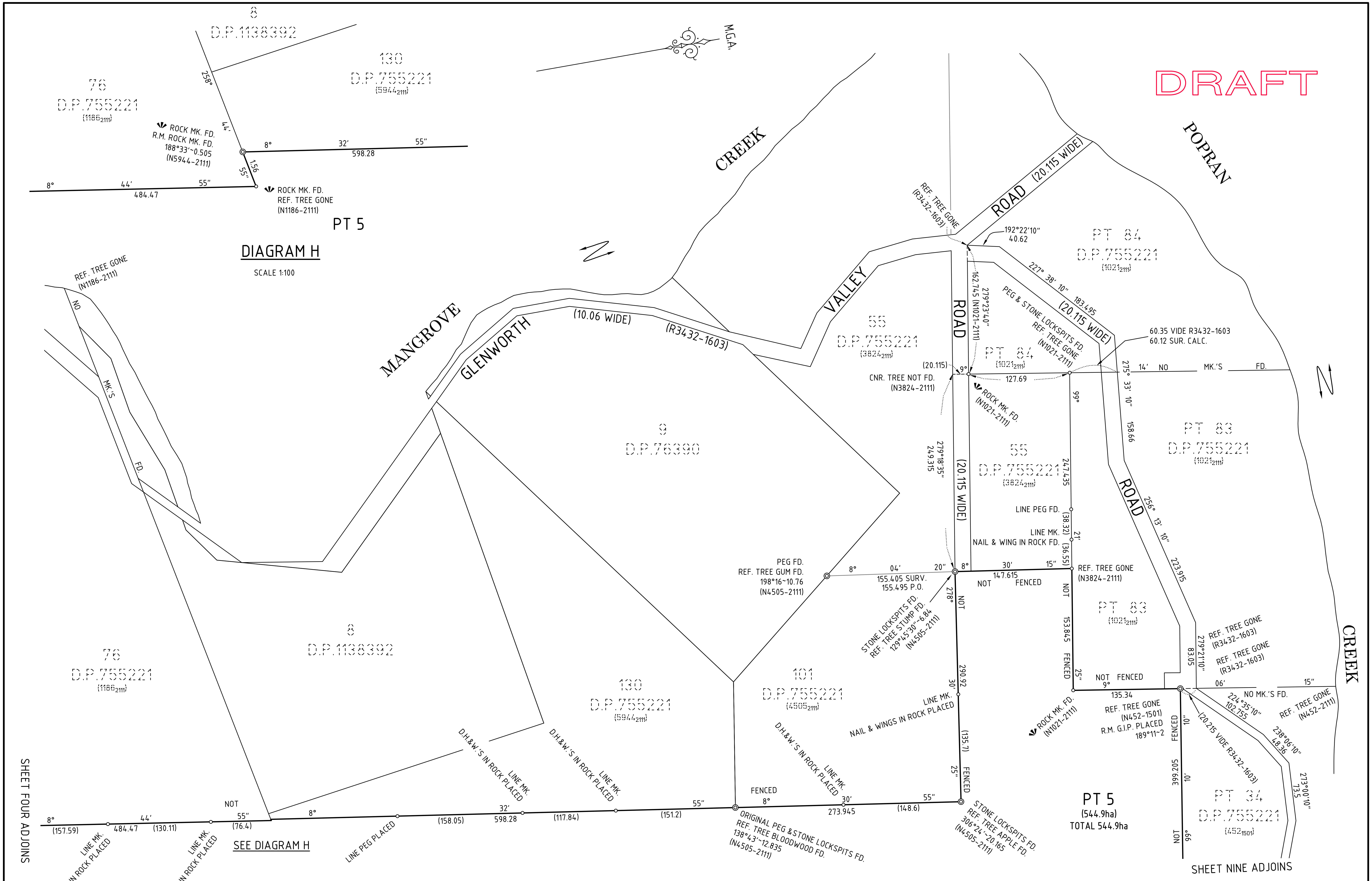




DRAFT

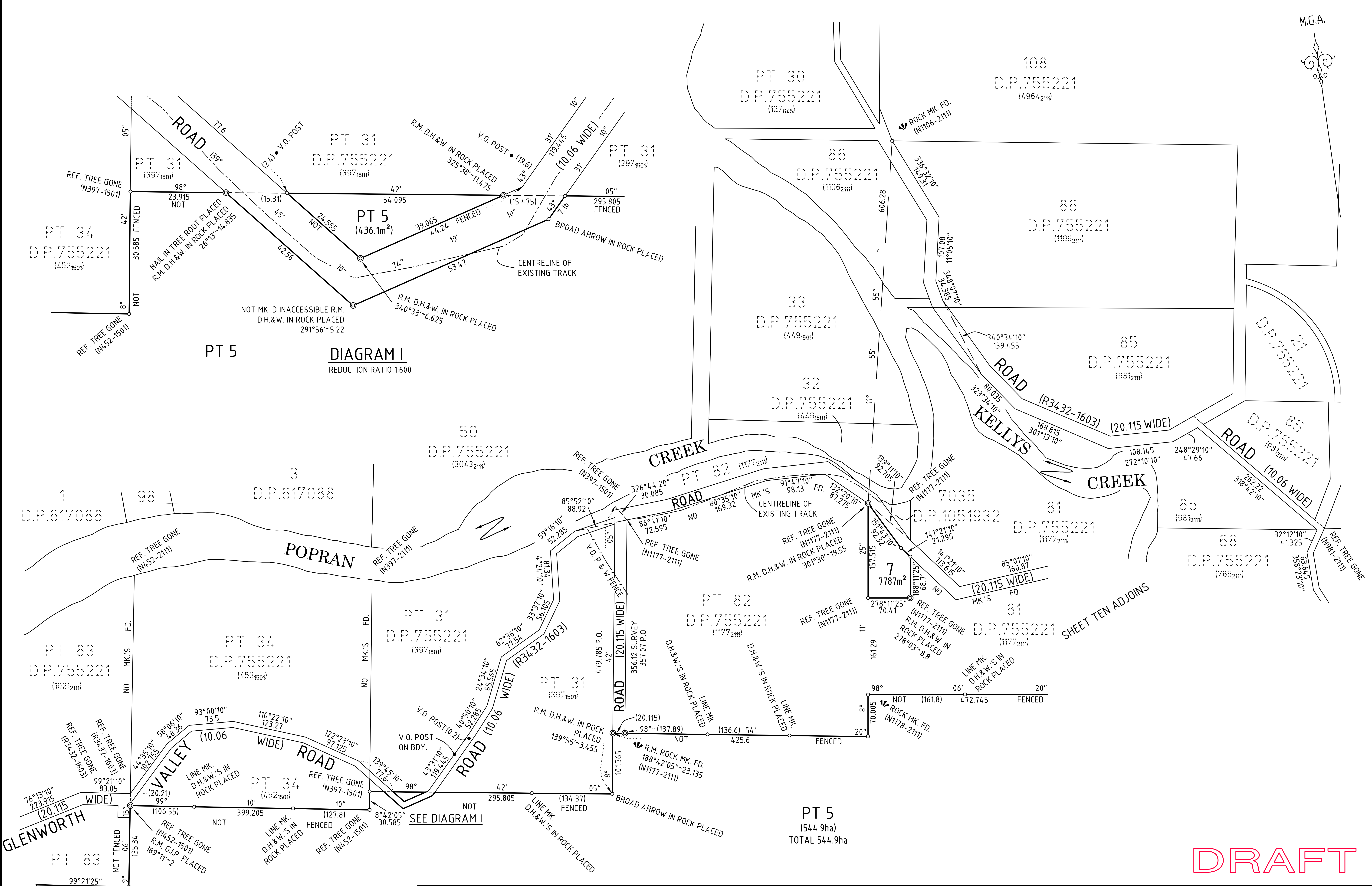
Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:4000	Registered	D.P.

10 20 30 40 50 Table of mm 90 100 110 120 130 140



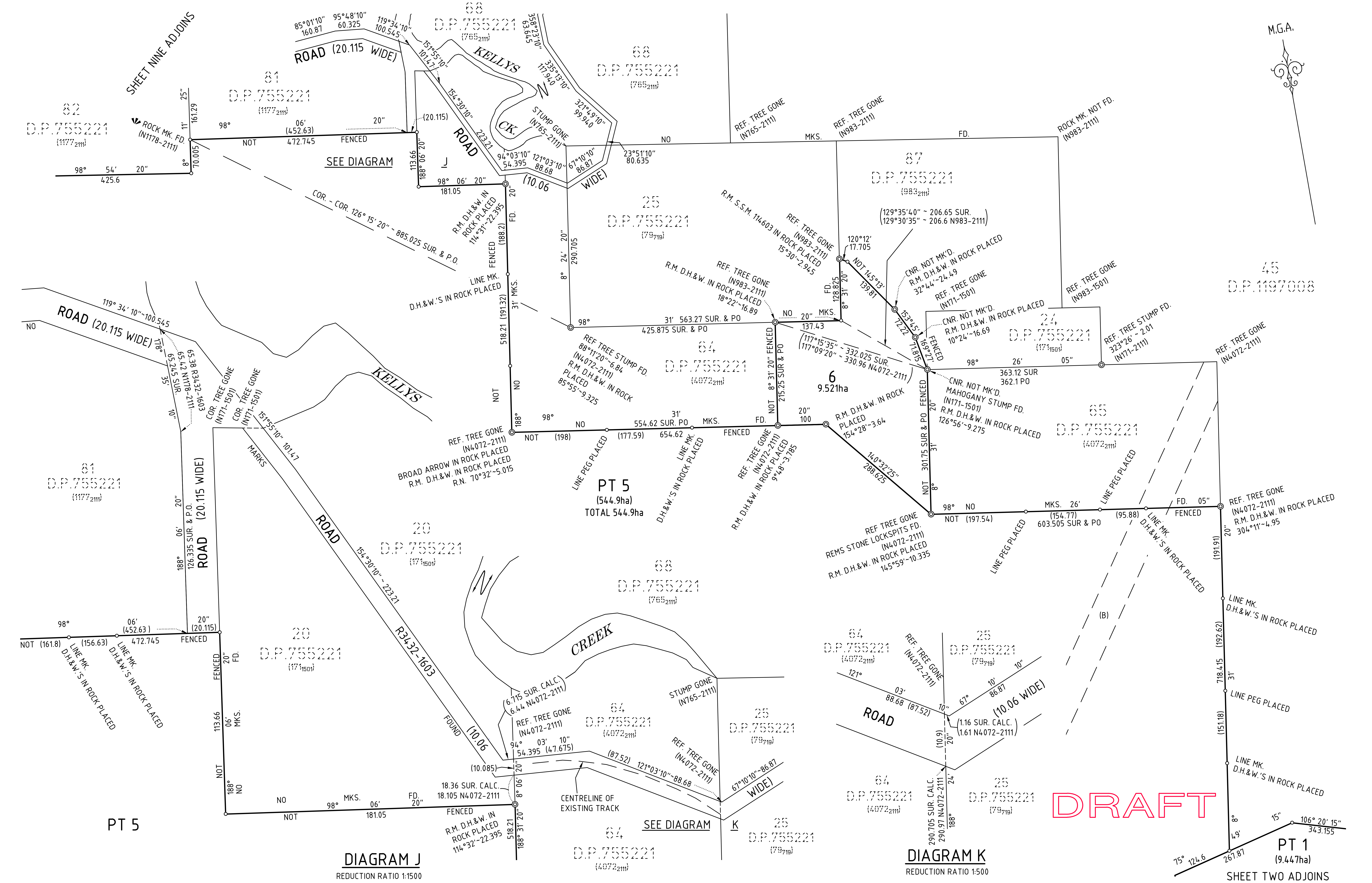
Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:3000	Registered	D.P.

10 20 30 40 50 Table of mm 90 100 110 120 130 140



Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:4000	Registered	D.P.

10 20 30 40 50 Table of mm 90 100 110 120 130 140



(B) EASEMENT FOR ELECTRICITY TRANSMISSION LINE 60 WIDE G.G. 10.09.1976 FOL. 3877 (CP8843-8844.3070)

Surveyor: ANTHONY JAMES OLIVER Date of Survey: Surveyor's Ref: 190299S-DP-001-B 2016M7100(428) PARTIAL SURVEY 2016M7100(1746) ADDITIONAL SHEETS	PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.	LGA: CENTRAL COAST Locality: GLENWORTH VALLEY, MOUNT WHITE AND WENDOREE PARK Subdivision No: N/A Lengths are in metres. Reduction Ratio: 1:5000	Registered	D.P.

PLAN FORM 6 (2013) WARNING: Creasing or folding will lead to rejection

DEPOSITED PLAN ADMINISTRATION SHEET

SHEET 1 OF 2 SHEET(S)

Office Use Only

Registered:

Title System: TORRENS

Purpose: SUBDIVISION

Office Use Only

D.P.

PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.

LGA: CENTRAL COAST

Locality: GLENWORTH VALLEY, MOUNT WHITE & WENDOREE PARK

Parish: COWAN

County: NORTHUMBERLAND

Crown Lands NSW/Western Lands Office Approval

I .... (Authorised Officer) in approving this plan certify that all necessary approvals in regard to the allocation of the land shown herein have been given.

Signature:..

Date:...

File Number:...

Office:...

Subdivision Certificate

THE PLAN IS EXEMPT FROM SUBDIVISION CERTIFICATE UNDER SECTION 23G (k) OF THE CONVEYANCING ACT 1919.

Survey Certificate

I, ANTHONY JAMES OLIVER

of ADW JOHNSON PTY LIMITED

P.O. BOX 3717 TUGGERAH NSW 2259

a surveyor registered under the *Surveying and Spatial Information Act 2002*, certify that:

~~*(a) The land shown in the plan was surveyed in accordance with the *Surveying and Spatial Information Regulation 2012*, is accurate and the survey was completed on~~ ....

*(b) The part of the land shown in the plan (~~*being~~/*excluding ^ ....PART OF LOT 5..................) was surveyed in accordance with the *Surveying and Spatial Information Regulation 2012*, is accurate and the survey was completed on,……………….. the part not surveyed was compiled in accordance with that Regulation.

~~*(c) The land shown in this plan was compiled in accordance with the *Surveying and Spatial Information Regulation 2012*.~~

Signature: .... Dated: ....

Surveyor ID: ....8357............

Datum Line: ....'X' - 'Y'...........

Type: ~~*Urban/~~*Rural

The terrain is ~~*Level-Undulating~~ / *Steep-Mountainous.

*Strike through if inapplicable.

^Specify the land actually surveyed or specify any land shown in the plan that is not the subject of the survey.

Statements of intention to dedicate public roads, public reserves and drainage reserves.

Plans used in the preparation of this survey ~~/ compilation~~

49-719	452-1501	1186-2111	5944-2111	6897-2111
53-2071	453-1501	1296-2111	6614-2111	7778-2111
70-2071	765-2111	1307-2111	6616-2111	8114-3070
70-3070	976-2111	1505-2111	6663-2111	8307-2111
79-719	981-2111	2122-3070	6664-2111	8843-3070
83-2111	982-2111	3824-2111	6665-2111	8844-3070
112-719	983-2111	3877-2111	6666-2111	R3432-1603
171-1501	1021-2111	4072-2111	6667-2111	R7857-1603
298-1501	1104-2111	4505-2111	6672-2111	R19911-1603
331-1501	1106-2111	4533-2111	6673-2111	R23786-1603
337-1501	1177-2111	4553-2111	6674-2111	DP25510
397-1501	1178-2111	5152-2111	6686-2111	DP40568

If space insufficient continue on PLAN FORM 6A SEE SHEET 2

Signatures, Seals and Section 88B Statements should appear on PLAN FORM 6A

SURVEYOR'S REFERENCE: 190299-DP-001-B

2016M7100(428) PARTIAL SURVEY, 2016M7100(1746) ADDITIONAL SHEETS

DEPOSITED PLAN ADMINISTRATION SHEET

Registered: Office Use Only

Office Use Only

D.P.

PLAN OF SUBDIVISION OF LOT 161 IN D.P. 755221, LOT 7020 IN D.P.1065007, LOT 7021 IN D.P.1114447, LOT 7310 IN D.P.1155132, LOT 7040 IN D.P.1116103, LOT 7027 IN D.P.1051931, LOT 7041 & 7042 IN D.P.1116109 AND LOT 2597 IN D.P.1205726.

This sheet is for the provision of the following information as required:

- A schedule of lots and addresses - See 60(c) *SSI Regulation 2012*
- Statements of intention to create and release affecting interests in accordance with section 88B *Conveyancing Act 1919*
- Signatures and seals- see 195D *Conveyancing Act 1919*
- Any information which cannot fit in the appropriate panel of sheet 1 of the administration sheets.

Subdivision Certificate No: . . . .

Date of Endorsement: . . . .

Plans used in the preparation of this survey / ~~compilation~~ (continued)

DP93604	DP641318	DP1052984
DP207158	DP644092	DP1060114
DP247984	DP653279	DP1065007
DP253871	DP720692	DP1100501
DP444175	DP727740	DP1114447
DP445868	DP749433	DP1116103
DP446135	DP802793	DP1116109
DP499011	DP814384	DP1138392
DP499014	DP823168	DP1155132
DP539733	DP828634	DP1152224
DP547622	DP843674	DP1173880
DP553372	DP844113	DP1197008
DP563601	DP853399	DP1205726
DP639733	DP856953	
DP641317	DP1051931	

CERTIFIED CORRECT FOR THE PURPOSES OF THE REAL PROPERTY ACT 1900

....

Kevin Thompson
SENIOR REGISTERED SURVEYOR
DEPARTMENT OF PRIMARY INDUSTRIES - LANDS

BY DELEGATION PURSUANT TO SECTION 180 OF THE CROWN LANDS ACT 1989 AND WITH THE AUTHORITY UNDERSECTION 13L OF THE REAL PROPERTY ACT 1900 FROM THE MINISTER ADMINISTERING THE CROWNS LANDS ACT 1989 ON BEHALF OF THE STATE OF NEW SOUTH WALES

STREET ADDRESSES OF ALL LOTS ARE NOT AVAILABLE

If space insufficient use additional annexure sheet

SURVEYOR'S REFERENCE: 190299-DP-001-B 2016M7100(428) PARTIAL SURVEY, 2016M7100(1746) ADDITIONAL SHEETS

Densification of the Greater Sydney Subspine Network for Roads and Maritime Services Western Sydney Infrastructure Plan Projects

Michael Dunn
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ABSTRACT

Spatial Services, a unit of the Department of Finance, Services and Innovation (SS-DFSI) has developed a network of Class A survey marks covering the Greater Sydney area. Known as the Greater Sydney Subspine Network (GSSN), it assists design, construction and maintenance of road and rail infrastructure projects covering large, linear areas. Roads and Maritime has broken down this network utilising static Global Navigation Satellite System (GNSS) methods to Class B standard in order to provide a control framework for multiple road upgrade projects in western Sydney. Given the unprecedented growth predicted in western Sydney over the coming decades and the thousands of direct and indirect jobs for the region created by the opening of the western Sydney airport at Badgerys Creek, Roads and Maritime has developed the Western Sydney Infrastructure Plan (WSIP), a 10-year plan to provide improved road transport capacity ahead of future traffic demand. Control surveys undertaken by Roads and Maritime, several by working with private industry from Roads and Maritime's panel of specialist providers, support this plan by providing a control framework that will be utilised throughout the various stages of each project, including ground modelling, utility investigations, construction and boundary surveys, and subsequent asset maintenance activities. The control framework provides a number of additional benefits, including preservation and reinstatement of survey control infrastructure, and a legacy control framework to benefit the greater community. Given the GSSN will be used to define GDA2020, Roads and Maritime control networks provide a direct linkage to GDA2020, allowing a very accurate set of transformation parameters to be developed. Future surveys connecting to Roads and Maritime networks, including those by traditional methods, will also provide a link to GDA2020. This paper outlines the extent of surveys undertaken by Roads and Maritime, the extent of the GSSN and plans for the development of regional subspine networks, and demonstrates the benefits of this approach now and into the future.

KEYWORDS: *Survey control, GNSS, GDA2020, GSSN, WSIP.*

1 INTRODUCTION

The Greater Sydney Subspine Network (GSSN) was established by Spatial Services, a unit of the Department of Finance, Services and Innovation (SS-DFSI). The network provides over 300 Class A marks coordinated via a single static Global Navigation Satellite System (GNSS) adjustment extending throughout the Greater Sydney area. Coordinates are based on the GDA94(1997) realisation of the Geocentric Datum of Australia 1994 (for more details see Janssen and McElroy, 2010) with readjusted values published in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013) in June 2013.

GNSS control surveys established for large road and rail infrastructure projects can connect to and break down the GSSN to provide a highly accurate network over the project area. A major benefit of this are site-specific adjustments that are not subject to localised distortions which can occur when establishing a datum based on connection to local permanent survey marks spanning multiple SCIMS adjustments.

Throughout 2015 and 2016, the Roads and Maritime Surveying Section established an extensive network of large scale GNSS control surveys in western Sydney connected to the GSSN. These surveys provide a control framework for many Western Sydney Infrastructure Plan (WSIP) projects. WSIP is Roads and Maritime's 10-year plan to provide improved road capacity by building many new and upgraded arterial roads in western Sydney to meet anticipated future demand (Roads and Maritime, 2017).

Although undertaken in isolation, many WSIP projects overlapped and were reliant on a consistent set of coordinates on common marks. Connection to an overarching framework of highly accurate control marks ensured a smooth transition between adjacent projects.

Survey control networks established for each WSIP project are used throughout the project life cycle, from initial ground modelling and utility location surveys, through to construction and boundary surveys, and subsequent asset maintenance activities. The resulting network provides a legacy for all future surveys and land development projects. Networks similar to the GSSN are being established by SS-DFSI throughout regional NSW to service areas with a high volume of infrastructure development. The methods utilised by the Roads and Maritime Surveying Section in Sydney may be applied to regional projects.

This paper provides an overview of the GSSN utilised by Roads and Maritime. It also shows the extent of the many survey control networks undertaken to service the WSIP projects. The paper highlights Roads and Maritime's specialised knowledge in high-order control surveys, its ability to work collaboratively with private industry to deliver results, application of Surveyor General's Direction No. 12 (DFSI Spatial Services, 2012) to ensure recognition in the SCIMS database and the future expansion of subspine networks throughout NSW.

2 GREATER SYDNEY SUBSPINE NETWORK (GSSN)

The GSSN was developed by SS-DFSI to address the need for a homogeneous network of Class A marks covering the Greater Sydney area to service large scale infrastructure projects. The network provides over 300 Class A marks coordinated via a single static GNSS adjustment. Coordinates are based on GDA94(1997) with readjusted values published in SCIMS in June 2013.

Observations forming the GSSN were used to define GDA2020, however the release of preliminary coordinates pre-date those that will be published in GDA2020. The network is made up of a combination of existing SCIMS marks and marks placed especially for the purpose of the subspine network. All marks have been specifically selected or placed based on fulfilling the requirements for accessibility, GNSS suitability, safety (i.e. high road traffic areas) and survey station density.

At the time of writing this paper, it was not possible to search explicitly for GSSN marks. However SS-DFSI proposes to incorporate an 'infrastructure layer' within SCIMS containing

subspine marks both in Sydney and regional NSW. A KML file appears the most logical way to publicly distribute this information. However, if a particular GSSN mark is known, the coordinates are readily available through SCIMS. As an example, the SCIMS search for GSSN mark PM42445 is shown in Figure 1.

SURVEY MARK

Mark	Name	Alias
PM 42445		n/a

Status	Date	Comments
	n/a	n/a

Location	Monument	Date Placed	Placed By
GROUND LEVEL	UNKNOWN	n/a	0

GDA94

Easting	Northing	Zone	Latitude	Longitude
295246.438	6261562.394	56	-33° 45' 58.58036"	150° 47' 20.57838"

Class	Order	Positional Uncertainty	Local Uncertainty	GDA Updated
A	1	n/a	n/a	18-FEB-2014

Source	Type	Method	Date issued	Issued By
235356	ADJUSTMENT	GEOLAB	28-JUN-2013	MICHAEL LONDON

Previous Reference	Location	File Number
n/a	n/a	n/a

Comments
GREATER SYDNEY SUBSPINE TRANSACTION #100093

MGA Combined Scale Factor	MGA Convergence
1.000106	-1° 13' 45.44"

AusGeoid09
23.442

AHD71

Height
41.494

Class	Order	Positional Uncertainty	Local Uncertainty	AHD Updated
B	2	n/a	n/a	18-FEB-2014

Source	Type	Method	Date issued	Issued By
235356	ADJUSTMENT	GEOLAB	28-JUN-2013	MICHAEL LONDON

Previous Reference	Location	File Number
n/a	n/a	n/a

Comments
GREATER SYDNEY SUBSPINE TRANSACTION #100093

Figure 1: SCIMS search for Greater Sydney Subspine Network (GSSN) mark PM42445.

SS-DFSI is developing additional Class A subspine networks in regional NSW to support areas with a high volume of infrastructure development. SS-DFSI will also support regional infrastructure projects by creating ad-hoc networks (see section 5).

The GSSN network established during the period of Roads and Maritime control surveys is shown in Figure 2.

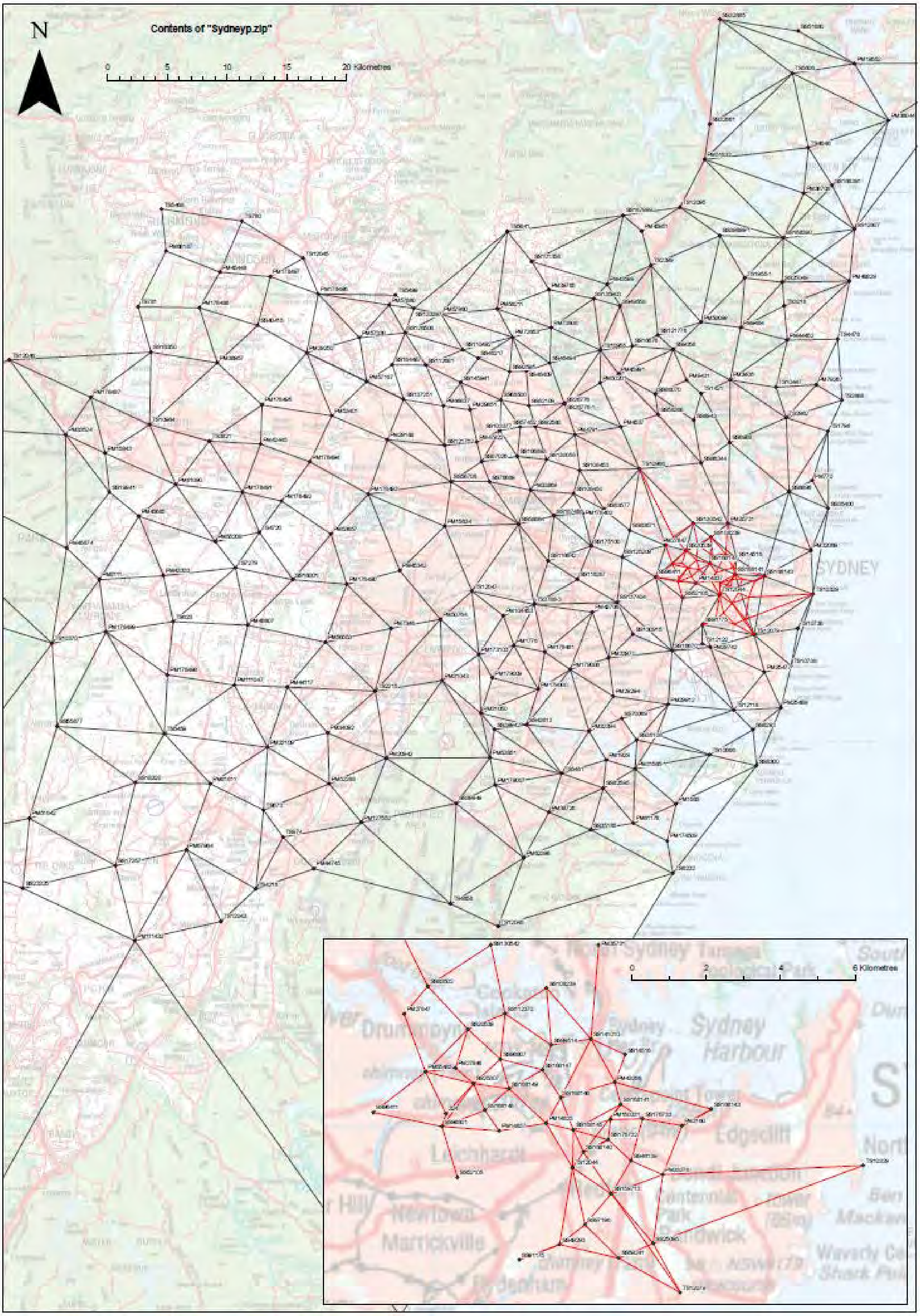

Figure 2: Greater Sydney Subspine Network (November 2016).

The network incorporates several CORSnet-NSW stations (Janssen et al., 2016; DFSI Spatial Services, 2017). While the GDA94(2010) coordinate values for these Continuously Operating Reference Stations (CORS) obtained via Regulation 13 certification currently do not match those published in SCIMS (Janssen and McElroy, 2010), the modernised Australian datum (GDA2020) will provide a homogenous set of coordinates for both CORS and ground control marks. The current differences in coordinate values between SCIMS and CORSnet-NSW are shown in Table 1 and agree with the known distortions across the State (e.g. Haasdyk et al., 2014; Janssen et al., 2016).

Table 1: Coordinate comparisons of CORSnet-NSW stations vs. SCIMS.

		Coordinates – MGA Zone 56					**Difference: SCIMS - CORS**	
Mark	**Name**	**Easting (m)**	**Northing (m)**	**C**	**O**	**Source**	**ΔE (mm)**	**ΔN (mm)**
TS12043	Menangle	291928.734	6221563.023	A	1	235356	-2	-51
TS12044	Chippendale	333655.813	6248928.970	A	1	235356	+21	-33
TS12045	Mulgrave	298805.017	6277143.353	A	1	235356	+18	-11
TS12046	Springwood	274216.880	6268603.676	A	1	235356	-19	+11
TS12047	Villawood	312919.848	6249236.646	A	1	235356	+18	-23
TS12048	Waterfall	315117.373	6221146.449	A	1	235356	+20	-27
TS12079	UNSW	336547.298	6245564.220	A	1	235356	+28	-39
TS12095	Cowan II	330336.155	6281393.630	A	1	235356	+25	-22
TS12118	Port Botany	334826.525	6239282.703	A	1	235356	+20	-47

The GSSN was developed ahead of GDA2020 in order to address the need for a homogeneous network of Class A marks covering the Greater Sydney area to serve large scale infrastructure projects. Roads and Maritime surveys continually found excellent agreement between the published SCIMS values of all GSSN marks. Table 2 shows coordinate differences when all WSIP control surveys were merged into a single minimally constrained adjustment.

Table 2: Coordinate comparisons of western Sydney GSSN stations in a minimally constrained adjustment.

Mark	**C**	**O**	**ΔE (mm)**	**ΔN (mm)**	**Mark**	**C**	**O**	**ΔE (mm)**	**ΔN (mm)**
TS623	A	1	-4	-8	**PM45685**	A	1	+11	-6
TS2215	A	1	+5	+4	**PM48807**	A	1	Fixed	Fixed
TS3821	A	1	+23	-7	**PM53657**	A	1	+23	+6
TS4720	A	1	+2	+3	**PM55208**	A	1	+14	+4
TS5459	A	1	+18	-16	**PM56553**	A	1	-4	+3
TS7279	A	1	+3	+10	**PM57330**	A	1	+9	-9
TS10964	A	1	+28	-13	**PM57880**	A	1	-4	+6
TS12045	A	1	Fixed	Fixed	**PM61090**	A	1	+19	-5
PM7346	A	1	-2	-1	**PM81811**	A	1	+5	-13
PM8111	A	1	+5	-15	**PM111047**	A	1	+13	0
PM15843	A	1	+22	-3	**PM178490**	A	1	+16	+3
PM20942	A	1	+6	-6	**PM178491**	A	1	+12	-3
PM22109	A	1	+27	+2	**PM178492**	A	1	+9	+3
PM29250	A	1	+3	-15	**PM178493**	A	1	+7	-5
PM31043	A	1	+6	+4	**PM178494**	A	1	+9	+4
PM31050	A	1	+16	+13	**PM178496**	A	1	+12	-7
PM34092	A	1	+12	-13	**PM178498**	A	1	+5	-8
PM42445	A	1	+18	+2	**PM178499**	A	1	0	-12
PM43333	A	1	+10	-10	**SS16001**	A	1	+9	+7
PM44117	A	1	+3	+6	**SS18320**	A	1	+13	-13
PM45342	A	1	+9	+8	**SS19941**	A	1	+17	-9
					SS40415	A	1	+14	-14

3 ROADS AND MARITIME CONTROL SURVEYS

3.1 Application of Surveyor General's Direction No. 12 and General Roads and Maritime Procedure

Roads and Maritime has undertaken many GNSS control surveys for western Sydney infrastructure projects. A sample survey is provided in section 3.2. The compilation of all western Sydney control surveys is outlined in section 3.3.

Ultimately, all marks surveyed by static GNSS will be recognised in the SCIMS database at Class B standard. In order to ensure this standard of recognition, all of the procedures outlined in Surveyor General's Directions No. 12 – Control Surveys and SCIMS (SGD12 – see DFSI Spatial Services, 2012) are followed. SGD12 provides a guide as to what SS-DFSI requires before control survey information can be placed on public record in the SCIMS database.

The initial planning stage involves a comprehensive search of the SCIMS database within the project extent. At this point, reference is also made to the Greater Sydney Subspine Network (GSSN) to determine suitable marks to be included in the survey. A field reconnaissance is undertaken by Roads and Maritime. The status of marks destroyed or disturbed is reported via SS-DFSI's *Survey Mark Status Report Form*.

SGD12 emphasises the importance of consultation with SS-DFSI regarding selection and placement of marks, network design, station density, observation and processing techniques, levels of redundancy, equipment and presentation of results. These are all critical if the network is to be recognised to the desired Class in SCIMS. Roads and Maritime control surveys aim for and achieve horizontal Class B status. The requirements outlined in ICSM's Standards and Practices for Control Surveys (SP1), version 1.7 (ICSM, 2007) for static GNSS surveys are fulfilled.

When selecting the GSSN marks, the principle of working from the whole to the part is imperative. GSSN marks are selected such that the project is wholly contained within a polygon joining adjacent GSSN marks surrounding the project area. Figure 3 shows the marks selected for survey supporting the proposed upgrade of a section of Cambridge Avenue and Moorebank Avenue, Casula to Moorebank. The survey extents are shown in red.

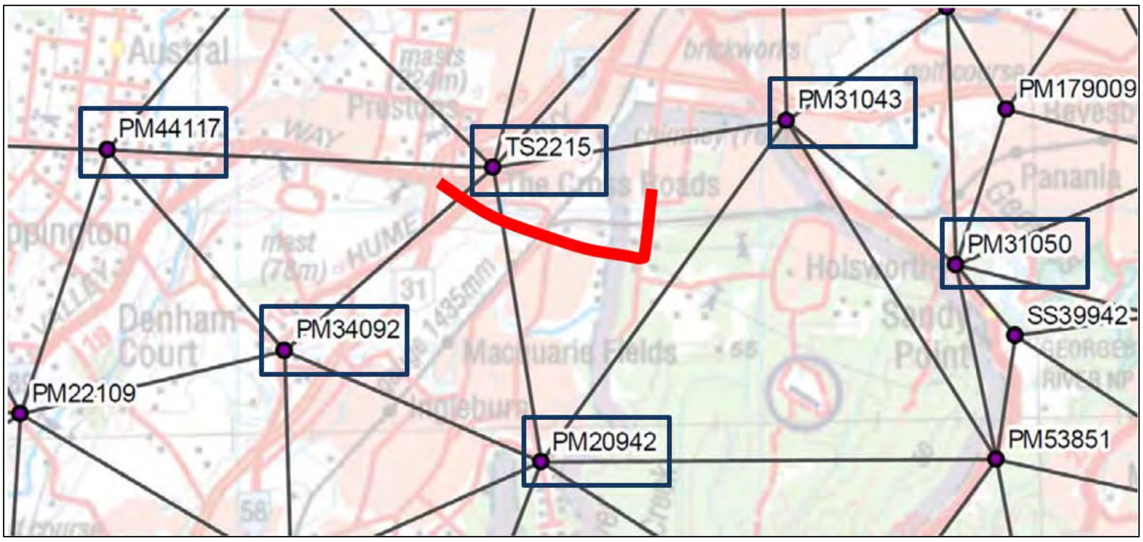

Figure 3: Greater Sydney Subspine Network marks selected for survey of Cambridge Ave and Moorebank Ave.

Network design requires a balanced connection to the GSSN marks surrounding the network. Baselines join all adjacent marks. Redundancy exists within each network, however the network is not over-observed.

Some control surveys have been undertaken internally by Roads and Maritime survey teams. However, closer collaboration with the private sector has allowed Roads and Maritime to utilise a larger pool of resources and build up the industry's capability in this area of specialisation. Private industry partners performing static GNSS control surveys initially work under the direction of Roads and Maritime's Senior Surveyor (Control Surveys). This ensures private industry partners are aware of the field and office procedures to fulfil the requirements of both SGD12 and SP1 to achieve Class B results. RINEX format data has been used where a combination of GNSS manufacturer's equipment is employed.

A minimally constrained least squares network adjustment is the first step in any network adjustment with one GSSN mark held fixed in position and level (height), followed by an assessment of the integrity of the network. Baselines with large residuals may be reprocessed in the GNSS processing software in an attempt to improve the quality of the baseline before being retested in the unconstrained network adjustment.

The minimally constrained adjustment tests the statistical Class of the survey by analysis of the relative error ellipses between marks. Adjustment software such as Compnet used by Roads and Maritime will perform a Class test at each selected Class. However, as far as recognition in the SCIMS database is concerned, the statistical Class result is meaningless if procedures outlined in SP1 regarding field procedures, equipment and processing techniques have not been fulfilled.

Once the minimally constrained adjustment has been completed, a comparison is made to the GSSN marks. It is often useful to apply a standard deviation to the Easting, Northing and level (height) values to determine a 'best-fit' to the marks. Roads and Maritime surveys have revealed excellent agreement between GSSN marks (see Table 2). All GSSN marks are held fixed in the final constrained adjustment.

Infill traverse surveys undertaken during ground modelling surveys include differential levelling throughout the project extents. Thus the definition of the vertical datum at the GNSS control survey stage is not critical. However, the final constrained GNSS adjustment holds fixed levelled, established marks, showing general agreement.

Investigation surveys for Roads and Maritime are undertaken under Specification G73 *Detail Survey*. The recently released Edition 3 contains detailed requirements and checklists to ensure control surveys are undertaken to achieve Class C and differential levelling achieves Class LC standards as defined by SP1 version 1.7.

Furthermore, Roads and Maritime Specification G71 *Construction Surveys* (currently under review) establishes a Survey Mark Register for all control and cadastral reference marks. This register tracks the status of all marks within the construction zone. Ground modelling surveys populate this register with control marks prior to construction.

Critical to acceptance of the survey in SCIMS is the Survey Report with a template located on the SS-DFSI website. The Survey Report includes the checklist provided in SGD12 and is accompanied by all survey data relating to the project.

3.2 Sample Survey: The Northern Road (Stage 4), Luddenham to South Penrith

The Northern Road is being upgraded to a dual carriageway major arterial road between Oran Park and South Penrith. Control surveys have been undertaken over four sections to date. The northern-most section between Luddenham and south Penrith involved a survey undertaken by Roads and Maritime surveyors with infill traversing and levelling performed by several Roads and Maritime private industry partners. The GNSS control network extent is shown in Figure 4. GSSN marks selected to define the datum are highlighted.

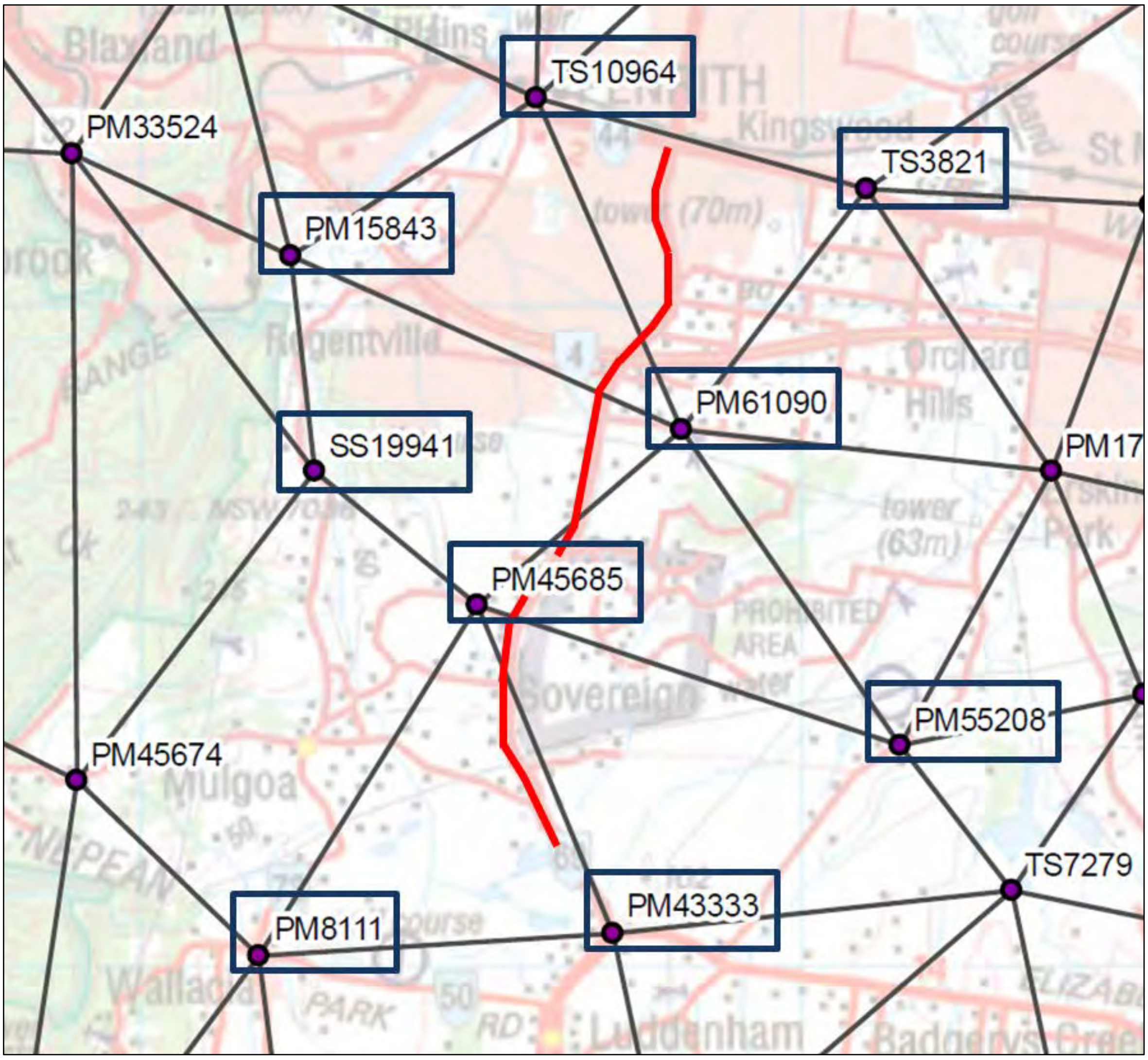

Figure 4: Static GNSS control survey network for The Northern Road, Luddenham to South Penrith.

The static GNSS control survey network, approved by SS-DFSI and surveyed by Roads and Maritime is shown in Figure 5. It should be noted that there are no baselines observed between GSSN marks, which is as per SS-DFSI instruction. Whilst these baselines may improve the statistical analysis of the unconstrained adjustment, their absence makes no difference to the final coordinates when all GSSN marks are held fixed. This network provides the framework on which all subsequent surveys will be based.

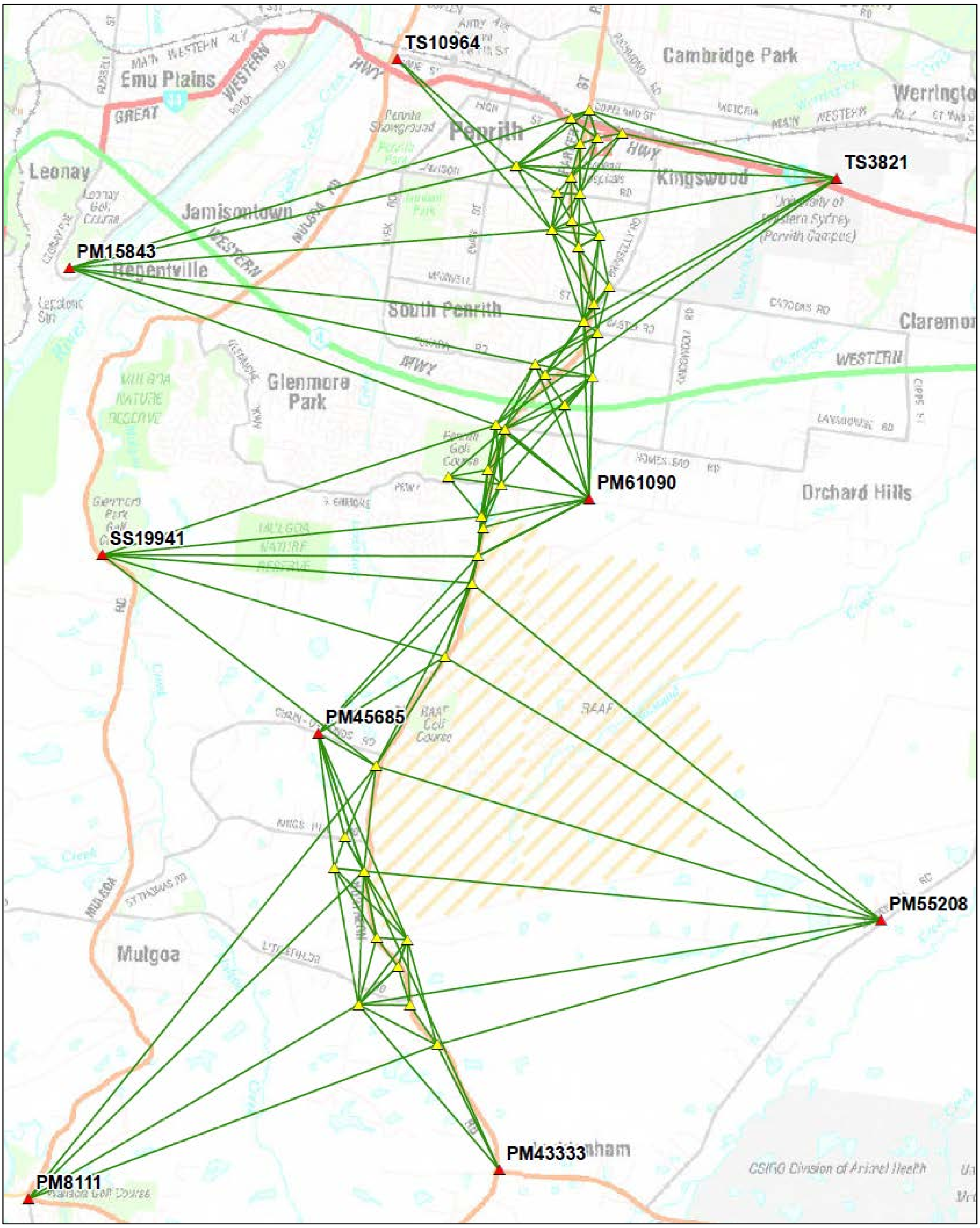

Figure 5: Static GNSS control survey network for The Northern Road, Luddenham to South Penrith.

The network was densified by traversing during the ground modelling survey. The traverse adjustment holds fixed for position only those marks coordinated by the GNSS adjustment. All other SCIMS marks, regardless of their Class and Order, are floated in the adjustment. All new traverse stations were differentially levelled with 2-way levelling runs to Class LC standard. The vertical adjustment is based on a best fit to local SCIMS values. The infilled traverse network is shown in Figure 6. Marks from the static GNSS adjustment are marked.

Figure 6: A section of infill traverse control for The Northern Road at South Penrith.

3.3 Combined Roads and Maritime Networks

The case study outlined in section 3.2 is one of many control surveys undertaken by Roads and Maritime in western Sydney with each survey following the same process, and several of these surveys overlapping. Connection to, and adoption of, the homogeneous GSSN meant a smooth transition between projects, free of the distortions and steps that often occur when networks adopt local SCIMS control across multiple adjustments.

Roads and Maritime has to date completed the following static GNSS networks across western Sydney linked to the GSSN:

- The Northern Road, Stages 1-4, Oran Park to South Penrith.
- Mamre Road, Kemps Creek to Claremont Meadows.
- The Horsley Drive, Horsley Park to Bossley Park.
- Archbold Road, Minchinbury to Horsley Park.
- Bandon Road, Vineyard.
- Fifteenth Avenue, Austral to Hoxton Park.
- Cambridge Avenue, Casula to Moorebank.

Figure 7 shows the combined static GNSS surveys undertaken and managed by Roads and Maritime supporting the Western Sydney Infrastructure Plan, with connection to the GSSN. At the time of writing this paper, the M12 Motorway joining the M7 Motorway to The Northern Road, incorporating an upgrade to Elizabeth Drive, has been announced. A proposed network has been approved by SS-DFSI and is included in Figure 7. This network will be surveyed in early 2017.

As demonstrated above, many GSSN marks are common between projects with an overlap on some projects, including Mamre Road and the proposed M12 Motorway/Elizabeth Drive surveys, and the four stages of The Northern Road. Some statistics regarding the 10 survey networks shown in Figure 7 include:

- Number of GSSN marks utilised: 43.
- Number of SCIMS marks coordinated by GNSS relative to GSSN: 220.
- Number of additional construction-quality marks coordinated by GNSS relative to GSSN: 40.
- Number of SCIMS marks proposed to be coordinated on M12 survey: 49.
- Number of approved survey marks to be placed and coordinated on M12 survey: 23.

At the time of writing, all surveys with the exception of M12 Motorway/Elizabeth Drive have been submitted to SS-DFSI. An update of the SCIMS database will most likely take place following the release of GDA2020.

Unlike the GSSN observations undertaken by SS-DFSI, Roads and Maritime GNSS observations have not been utilised to define GDA2020. However, the surveys now provide a direct linkage to GDA2020 and allow an accurate set of transformation parameters to be developed. This is also the case for the infill traversing marks that have further broken down the static GNSS control networks.

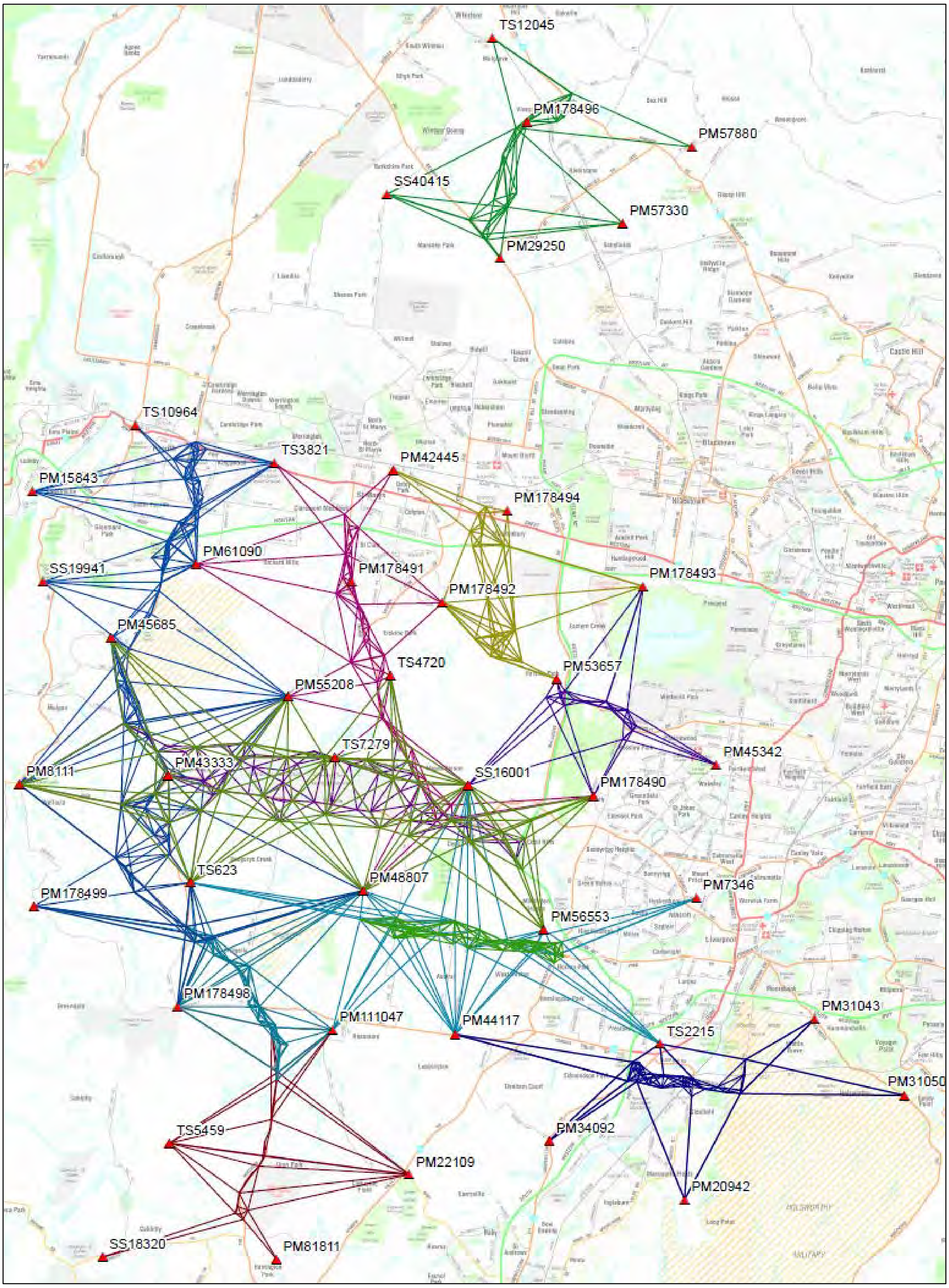

Figure 7: Combined static GNSS surveys connecting to the GSSN to support WSIP projects.

4 BENEFITS OF THIS APPROACH

There are many benefits to the approach taken by Roads and Maritime in western Sydney. Firstly, a homogeneous dataset of control coordinates exists across western Sydney, which is used as the basis for multiple infrastructure projects. Each network connects to the local datum, but each is free from the distortions that exist across multiple adjustments. Projects that overlap have a seamless transition between coordinates adopted at each site which is especially critical during the construction stages of these projects.

Secondly, it is anticipated that many survey control marks will be destroyed during construction. Roads and Maritime treats the preservation of survey infrastructure very seriously and has worked extensively with SS-DFSI to develop procedures in this regard. Having the original control marks directly linked to the overall GSSN will allow replacement control marks to be coordinated relative to the same datum, thus maintaining the integrity of the post-construction network.

Thirdly, all future surveys that link to the extensive network of marks will have a direct link to the marks used to define GDA2020. Whilst the construction phase of each project will maintain the coordinate system established during the initial stage of the project, SS-DFSI will eventually upgrade all SCIMS marks to GDA2020 coordinates. The marks coordinated by Roads and Maritime will have very accurate GDA2020 coordinates.

Finally, Roads and Maritime will leave a legacy framework following construction of these projects that will be used for future asset maintenance activities, future land use projects and will benefit the greater surveying community.

5 REGIONAL SUBSPINE NETWORKS

SS-DFSI is developing additional Class A subspine networks in regional NSW to support areas with a high volume of transport infrastructure development. SS-DFSI is also able to support regional projects on an ad-hoc basis. For example, south of Sydney observations for the South Coast A1 network from Sydney to the Victorian border have been reduced and adjusted by SS-DFSI. The results of this survey will soon be released publicly.

North of Sydney the network expands through the Central Coast and Hunter region. Observations are still being collected with an adjustment pending. Eventually the network will extend through the North Coast to the Queensland border. Figure 8 shows the network covering the NSW Central Coast.

Eventually the subspine network will cover much of the state of NSW, with the density of marks being the greatest in metropolitan areas. The release of GDA2020 will mean all subspine network marks will have homogeneous coordinates compatible with the CORSnet-NSW network. CORS may then be considered part of the subspine network. Increased station density will provide greater redundancy when breaking down the subspine network, leading to increased accuracy. The added bonus for surveyors is that they will not need to occupy CORSnet-NSW sites – GNSS RINEX data for each CORSnet-NSW site during the observation periods may be downloaded upon returning to the office.

Figure 8: Class A subspine network throughout the Central Coast.

The methodology described in this paper may be used as a template for future large road or rail infrastructure projects. As the subspine network expands to cover much of NSW, a similar approach may be taken for regional projects over large areas or in areas with little survey control infrastructure.

6 CONCLUDING REMARKS

The GSSN is a fantastic initiative by SS-DFSI. Released prior to GDA2020 and based on a single adjustment, it provides the foundation for control survey networks established for large scale infrastructure projects. The Roads and Maritime Surveying Section has utilised this network for projects falling under Roads and Maritime's Western Sydney Infrastructure Plan, developed to address the unprecedented growth predicted in western Sydney over the coming decades and subsequent infrastructure upgrades.

Roads and Maritime has broken down the subspine network by static GNSS observations and adjustment to Class B standard to provide a homogeneous network of control marks for each WSIP project. Projects that are developed in stages, such as The Northern Road, and projects sharing common extents benefit from this approach as a common set of coordinates between projects is established.

Roads and Maritime has undertaken much of the control surveys using internal resources. However, Roads and Maritime has also engaged with private industry specialist partners to

deliver these projects. Working in collaboration with Roads and Maritime, companies have a clear set of procedures fulfilling the requirements of SGD12, which may be applied on future projects. SGD12 has been an invaluable tool for ensuring SS-DFSI is consulted throughout the planning and execution of these control surveys, allowing the surveys to be eventually recognised in the SCIMS database to the desired Class.

Roads and Maritime is aware of its responsibilities to maintain the integrity of survey control infrastructure throughout the construction stages of projects. By coordinating new and existing survey control marks relative to the subspine network at the start of a project, any marks that are destroyed may eventually be replaced with approved survey marks coordinated relative to the overall network.

By coordinating a large network of survey marks relative to the subspine network, a very accurate set of GDA2020 transformation parameters can be established. All future surveys that connect to Roads and Maritime surveys will benefit from these initial surveys. Networks established by Roads and Maritime provide a legacy framework for the greater community. These networks may be utilised for future land development projects, cadastral surveys and many more applications.

The future state-wide expansion of subspine networks will provide benefits to regional infrastructure projects as has been realised in western Sydney. The approach for western Sydney projects may be used as a guide for the establishment of control survey networks for future regional infrastructure projects.

ACKNOWLEDGEMENTS

Roads and Maritime acknowledges the work performed by its *Geospatial Surveys* panel members to complete the following control surveys: Jacobs Group (Australia) Pty Ltd (The Northern Road Stage 3), Cardno (NSW/ACT) Pty Ltd (Mamre Road), Positive Survey Solutions Pty Ltd (Fifteenth Avenue), and NSW Public Works Advisory (Cambridge Avenue). The author wishes to acknowledge the support of Roads and Maritime Project Surveyor Jason Phipps in planning and processing many of these projects, providing mentorship during the processing stage to several contractors and creating control survey drawings at the conclusion of each project.

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Building a Bridge with a House Slab

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ABSTRACT

This paper describes the investigation and assessment process involved in developing an innovative technique for upgrading a small causeway in rural NSW. The final result has improved the level of service of the roadway, maintained the existing creek habitat, improved fish passage through the structure and installed a fit-for-purpose low maintenance structure that will service the community for years to come. The engineering design, which is considered a first, utilises residential house (waffle pod) slab construction methods to install a low-cost, in-situ concrete culvert with a minimum of road closure time and disruption to the watercourse. In order to avoid a long detour, Council along with the construction contractor facilitated access for the local residents at all times. The methodologies described in this paper may provide a blueprint for Local Governments faced with similar causeway upgrade requirements into the future.

KEYWORDS: *Causeway repair, waffle pod slabs, rural road culverts*.

1 INTRODUCTION

This paper outlines the journey of the Greater Taree City Council design team in dealing with a small concrete causeway on Belbora Creek Road, in rural NSW. The solution involved residential house slab technology (waffle pods), and we believe this was the first time this system has ever been used to create an in-situ concrete box culvert (Figure 1).

Figure 1: Looking upstream over the new Belbora Creek Road causeway during a freshet.

While the end result appears deceptively simple, it meets many of the requirements we deal with every day. It fulfilled the requirements of those little catch phrases that abound in

Council offices. It was ‘cost effective’, ‘fit for purpose’, ‘environmentally friendly’ and ‘provides an improved level of service’ to those residents who use the road.

2 LOCATION, DEFINITIONS AND BACKGROUND

Belbora Creek Road runs north off The Bucketts Way between Taree and Gloucester. It services the areas between Belbora and Bundook. This road predominately services rural properties ranging from 40 to 200 ha, primarily grazing beef cattle. Belbora Creek Road is about 23 km east of Gloucester, heading north off the Bucketts Way towards Bundook. Causeway #1 is, as the name suggests, the first causeway on the road and is located 5 km north of The Bucketts Way (Figure 2).

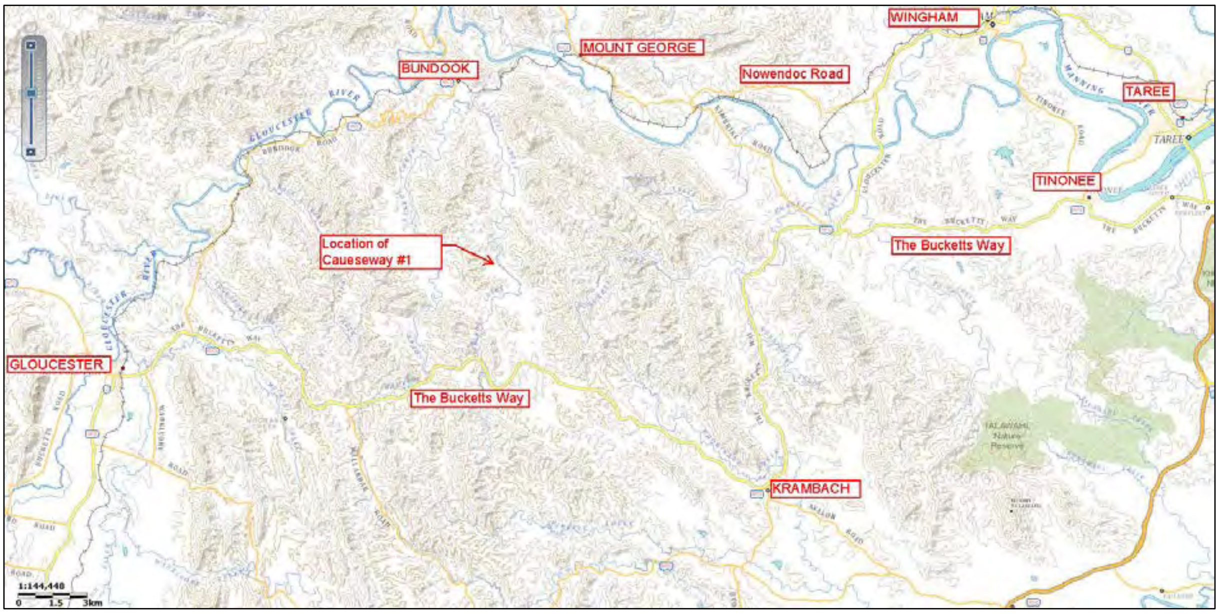

Figure 2: Locality map.

According to the English Oxford dictionary, a causeway is “a raised road or track across low or wet ground” (Oxford University Press, 2017). In this instance, the causeway acts as a road crossing Belbora Creek. As shown in Figure 3, Causeway #1 was a concrete slab near the level of the creek. A small Ø375 mm pipe is installed adjacent to the southern shore.

Older generations of the causeway can be seen off to the left of the current roadway. For many years and a number of generations of slabs, these low level causeways were considered ‘fit for purpose’ and provided the appropriate ‘level of service’ for this road. In this context, the level of service can be defined as “a qualitative measure representative of the operation of a road for a given traffic flow that takes into account a group of factors (speed, manoeuvre, safety, comfort and cost) that take place on it.”

Belbora Creek Road meanders from side to side across Belbora Creek, crossing the creek 10 times between The Bucketts Way and the rural area of Bundook. The road also wanders across the Local Government Area boundary passing between the former Greater Taree City Council and neighbouring former Gloucester Shire Council. However, Gloucester Shire Council had a higher priority on causeway upgrades in the early 1990s and upgraded all its causeways further along Belbora Creek Road, installing low-level bridges across the creek

(Figure 4). A bridge can be defined as "a structure that is built over a river, road or railway to allow people to cross from one side to the other" (Oxford University Press, 2017).

Figure 3: The original Causeway #1, looking north.

Figure 4: One of Gloucester Shire Council's upgraded causeway bridges (downstream side showing girders and mass concrete abutment).

These low-level bridges allow residents to cross the creek with a medium level of flow. Unfortunately, Greater Taree Council had different priorities and had not upgraded the very first causeway on the road. Residents from north of the causeway might negotiate the Gloucester Shire Council bridges all the way to #1 but were unable to negotiate the flooded causeway just short of The Bucketts Way, the main road to the rest of the district. Backtracking to exit the road via Bundook was a 30 km detour to get around the flooded causeway.

After Gloucester Shire Council completed its causeway upgrades, Causeway #1 ceased being a 'fit for purpose' rural creek crossing, providing a suitable 'level of service'. Despite absolutely no physical change to Causeway #1, users of this rural road reported that it had become a 'dangerous' and 'sub-standard' piece of infrastructure. The letters and complaints came in from the neighbouring Council's residents about this dramatic change in circumstances. The number of complaints waxed and waned with the weather as the level of inconvenience shifted. Particularly wet years in 2008 and 2009 brought the causeway upgrade to the Councils' Capital Works Programme in around 2010, marked by the Investigation & Design Project No 10/06.

3 BELBORA CREEK ROAD CAUSEWAY #1

3.1 General Information

Belbora Creek Road Causeway #1 was a concrete causeway on a rural road. Generally, these old causeways are made up of kerb or side log placed on either side of the roadway across the stream. A concrete slab, often un-reinforced, poured between these logs acts as the causeway deck. A small pipe may be fitted to the structure to take the low base flow that occurs between rainfall events. In the instance of Causeway #1, our investigations revealed that the causeway had three or four different generations of concrete poured at different levels making up the current structure and that the adjoining farmer would block the low flow pipe (with a sheet of ply or large rocks placed in the inlet) in order to maintain a higher level of water upstream of the causeway for irrigation purposes.

The Investigation & Design team was tasked with upgrading this causeway. However, flood disasters significant enough to attract disaster relief funding in 2011, 2012 and two in 2013 dropped the priority of the upgrade of this minor causeway. It is significant to note this creek crossing suffered very little damage in these major flood events. The design that had initially been started in 2010 did not really get underway until 2014.

The standard upgrade of a causeway is to rip it out, remove the existing causeway, install concrete box culverts and reinstate the causeway slab over the top of these box culverts. This was the initial design thinking and plans for a 5 x 2.4 m wide x 0.9 m high concrete box culvert had been prepared. The 45° skew of the road in relation to the creek made this a relatively expensive construction. In addition, Council construction crews were extremely wary of this as a methodology, having lost all their groundworks to rainfall events on several other comparable projects during the recent wet years.

The catchment of the causeway comprises approximately 4,453 ha of grazing country. The run-off was calculated using storage routing model (RAFTS) available in the DRAINS software and the flood modelling along the creek simulated using HEC-RAS. The depth of flow above the creek bed (and causeway) was assessed. The results of that assessment of the existing conditions at the crossing are shown in Table 1 for storms of various frequencies (Average Recurrence Interval – ARI).

Table 1: Assessment of the existing water flow conditions at the old Causeway #1.

ARI Storm Event	1- year	2-year	5-year	10-year	20-year	50-year	100-year
Depth of flow above creek bed (m)	2.05	2.56	3.10	3.38	3.66	4.00	4.30
Max. velocity of flow (m/s)	3.35	3.87	4.46	4.79	5.14	5.38	5.58

Clearly, significant flows occur at this location. The potential for Council to construct a bridge that remained flood-free at the ARI 100-year or 1% Annual Exceedance Probability (AEP) level was quickly discounted. A number of possible permutations and configurations of box culverts were assessed.

3.2 The Neighbour's Blueprint: Elegant Simplicity

Our investigation team went beyond the causeway and looked at how our neighbouring Council had addressed the identical problem just downstream of this site. Gloucester Shire

Council's Manager Technical Services, Gill Gendron, had designed and overseen the upgrade of the downstream causeways as low concrete bridges. These bridges, supported on girder logs, were considered, by the local residents, to be 'fit for purpose'. They provided conveyance over the creek during dry times and up to a low level of flow in the creek. These bridges were by no means flood-free and were cut off in larger flows once or twice each year, but this was only considered an inconvenience by the residents and an adequate 'level of service' was being provided. In comparison, Causeway #1 was cut off with minimal rainfall.

The design of these Gloucester Shire Council bridges was simple in structure, very cost effective at the time, required no specialised skills of the workforce and could easily be done using the machinery available at Gloucester Shire Council's disposal. As evident in Figure 5, the structure is made up of a series of bridge girder logs, of about 450 mm diameter, laid across the existing concrete causeway parallel with the flow of the stream. The logs are spaced at 1.5 m centres. This spacing enables a standard sheet of 19 mm form ply to be fixed between the logs and a 250 mm concrete slab to be poured over the logs. The form ply was simply left to rot and be washed away. The girders extend upstream and are fixed to a small series of piers installed in the stream bed. This tie-down on the upstream side of the structure prevents overturning of the entire structure in times of flood.

Figure 5: One of Gloucester Shire Council's upgraded causeways (upstream side showing extension of girders bolted to piers to prevent overturning).

The load of the structure and traffic is spread across a significant area under the girder logs, so the structural integrity of the original concrete causeways was not compromised. Clearly, these structures do not cater for runoff from any significant rainfall events in this catchment. They do, however, provide the locals with a 'level of service' that they are satisfied with. The structure does catch some debris as it floats downstream. However, apart from some rot or damage to the timber kerbs, these structures have stood the test of time to the satisfaction of the nearby residents and road users.

3.3 Design Challenge

The simplicity, low cost and ease of construction led our design team to reconsider its previous plans. Why not just copy the design of the neighbouring Councils' efforts?

- This design simply left the causeway in place. Even though this maintains the upstream habitat, these causeways are not considered fish friendly. Any modern construction would need approval of Fisheries NSW and fish passage would need to be part of the design.

- For longevity, the logs appear to have been treated. The green tinge is most likely Copper Chrome Arsenate (CCA). Leaving logs treated with these chemicals in a waterway would currently not be considered environmentally friendly.
- Girder logs have significantly increased in price. Logs that used to cost $500 would now cost in the range of $5,000 each, and a number of these would be required for a similar construction.
- Steel girders are currently relatively cheap! Could we simply replace the girder logs with steel beams?

A local consultant, Matrix Thornton Consulting Engineers (MTCE), was contracted to develop potential solutions, generally based on the Gloucester Shire Council example, and provide a suite of possible alternatives. Matrix Thornton evaluated 20 different options starting with girder logs, precast concrete beams, steel girders, box culverts, plastic arches (as used in sewer trenches) Bondek slab over concrete piers and waffle pod construction. The evaluation looked at construction costs of the various elements, the pros and cons for each and provided a method diagram for each of the options.

Being engineers who had designed many house slabs using the waffle pod technology, MTCE was comfortable with this method of construction and advocated for it, noting the potential negatives of some of the other alternatives and the specific advantages of the waffle pod product. MTCE noted particularly the speed of construction, the ability to leave the existing causeway in place, and minimal disturbance to the bed of creek. This gave Council officers the confidence to go on with the design process using the waffle pod system.

One significant challenge was the requirement for fish passage and requisite approvals of Fisheries NSW. Council could see that there was limited fish passage available on the other nine crossings on Belbora Creek Road and that all these crossings were between this causeway and the main river. Despite Fisheries NSW acknowledging this was the case, they explained that as each crossing was upgraded the ability for fish to access this upstream site would improve and this site would be ready when it finally happened (Figure 6).

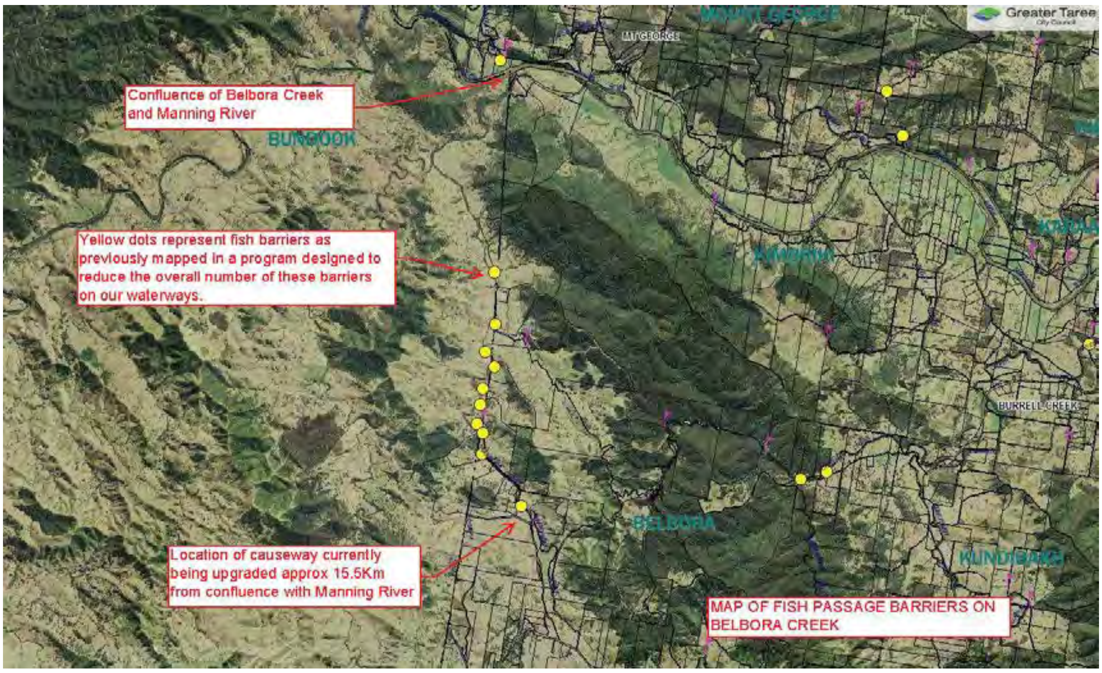

Figure 6: Fish barriers on Belbora Creek.

3.3.1 Waffle Pod Design

Waffle pods have been used in house construction for many years. First developed in South Australia to cater for the highly reactive clays of Adelaide, they have become widely used across Australia. The pods are boxes, 1.09 m square and varying in height. Depths of 175, 225, 300 and 375 mm are available to suit different ground conditions. They are made of light Styrofoam much like an old coolite surfboard. These boxes are basically void formers that suspend a concrete slab over the boxes between concrete beams that are formed between the pods. For houses this creates a stiff slab that is very strong.

For the waterway area of this project the pods were used as formwork that sat on the causeway and subsequently removed to create a skewed in-situ box culvert resting on the old causeway. The pods, stacked 2 high and 2 wide, created openings 2.18 m wide x 0.60 m high in the box culvert sections. Beyond the waterway, the pods were placed on the road and remain in place. The size of the pods was reduced as the road rose out of the causeway until it basically joined back to the level of the top of the new causeway. These approach pods significantly reduced the amount of concrete that was used in the abutments.

Removal of the pods from the causeway was considered a challenge. The pods were wrapped in plastic to prevent adherence of the concrete. MTCE engineers tested the varying amounts of solvent required to cause the pods to collapse primarily to prevent any excess solvent leaching into the river. Ultimately this solvent process proofed too slow for the contractors who found that mechanically breaking up the pods with a crow bar and shovel, and removing the material by hand, was the quickest and best method. The Styrofoam of these pods held together well, so there was very little breakout and the pieces were easily removed, collected and stored.

When these pods were first introduced into NSW, legislators foresaw a problem with the left-over sections of pods being a litter problem and determined a requirement whereby the manufacturer or supplier must collect all unused portions of the Styrofoam and remove it from site for re-use in the manufacturing process. On this project, all left-over foam was packaged into large bags and the supplier picked it up and returned it to the manufacturer for recycling.

3.3.2 Working with the Old Causeway

The design was by no means straight forward. The initial plan sets contained only three pages. The final plan set that went to tender was 20 pages long, containing numerous cross sections detailing the various stages of construction as foreseen when working with the existing structure (Figure 7).

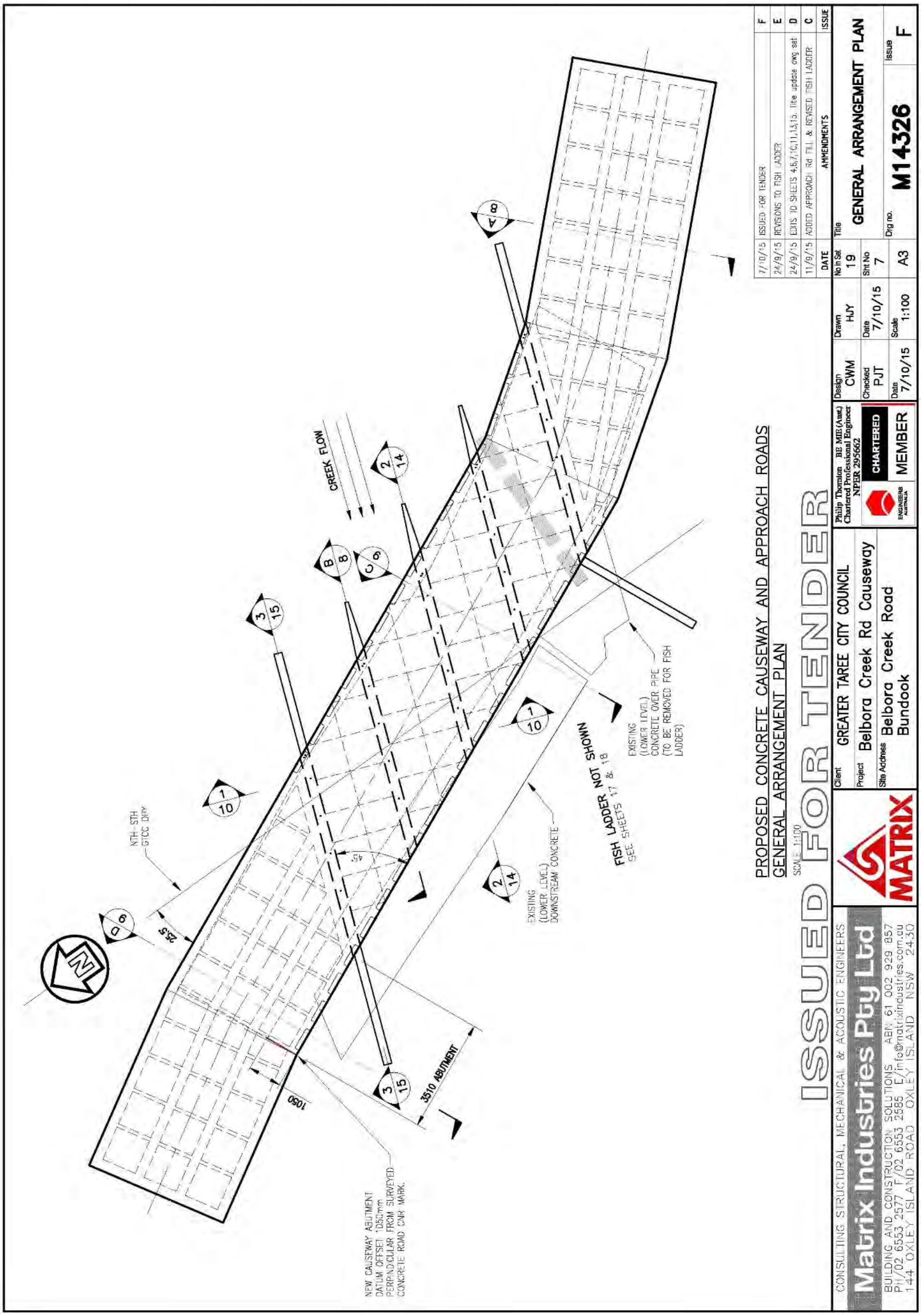

Figure 7: General arrangement of Waffle pods and concrete beams on the causeway and approach road slabs.

The old causeway was found to consist of a number of concrete slabs separated by gravel cobbles with the edges held in place by old timber (Figure 8).

Figure 8: Original causeway and edge timbers.

The design called for the removal of these old timbers. The voids left were replaced with concrete beams. These edge beams also gave the opportunity to widen the causeway so it would no longer be limited to the current width of causeway. The legs of the skew culvert rested on the old causeway. At these points, holes were drilled through the various slabs, shear connectors installed and grouted into the cobbles below.

The potential for overturning of this structure was similar to the Gloucester Shire Council causeway bridges. This was overcome by extending the legs of the culvert upstream and fixing them to the bedrock below. In order to ameliorate the debris problem noted previously on the downstream structures, these extensions were designed with a sloping face to act as deflectors channelling debris through or over the boxes (Figure 9). The debris is indicative of a flood that over-topped the causeway.

Figure 9: Upstream debris deflectors and northern approach slab.

Figure 10 illustrates the stages of construction for the new Causeway #1 structure.

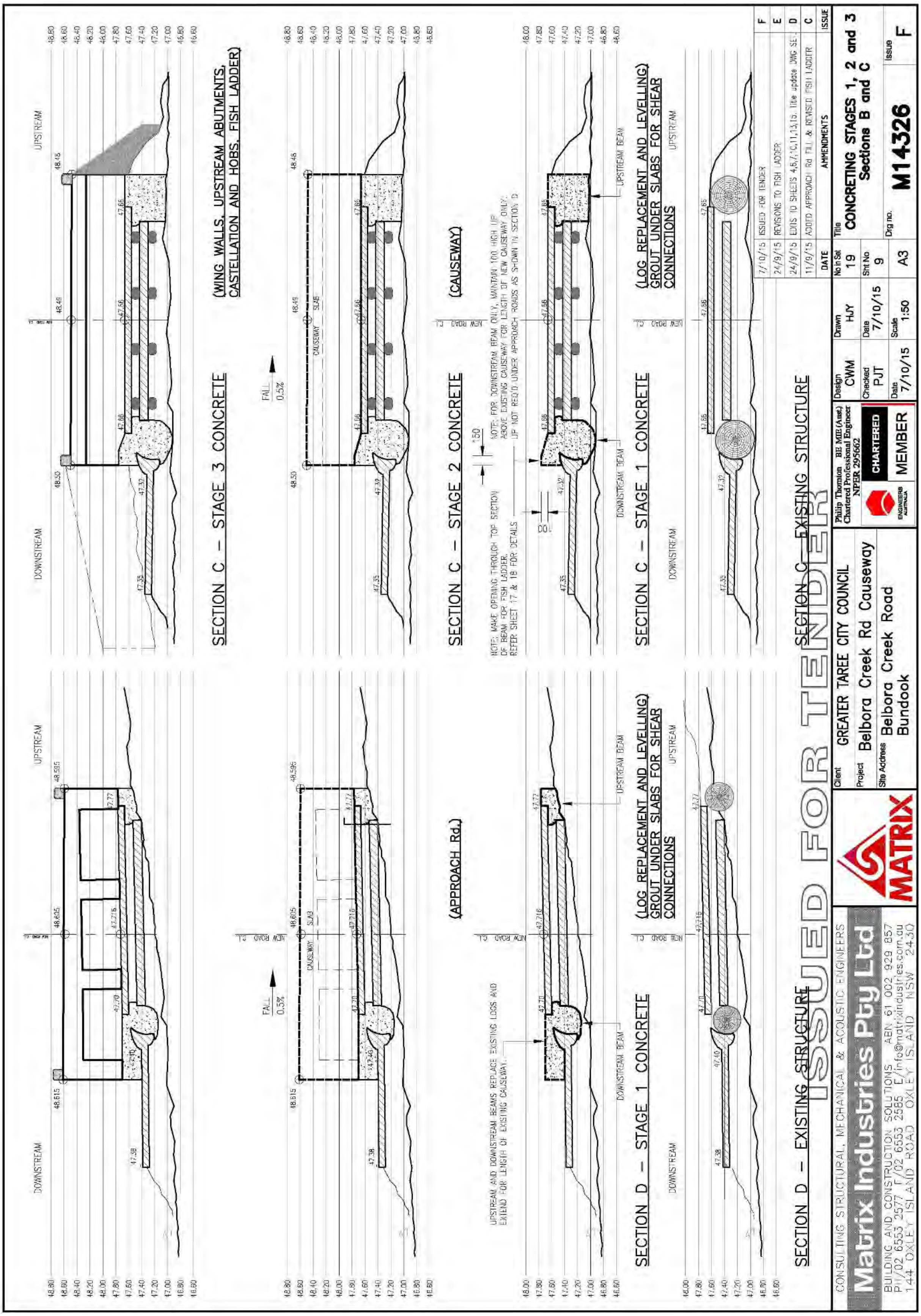

Figure 10: Stages of construction on existing causeway structure.

3.3.3 Fish Passage

One aspect of the design was to attain the approval of Fisheries NSW. Fish passage in engineering terms is a bit of an anathema because good hydraulics seldom equates to the provision of good fish passage. However, with some consideration and clever design it was achieved relatively simply.

In Hydraulics 101, basically there are two types of flow: subcritical flow and supercritical flow. In these terms, good fish passage requires maximum subcritical flow and minimum lengths and drops of supercritical flow.

Consider a causeway that has a cross fall with the flow, i.e. the high side on the upstream edge. To most people this is the most logical layout, however, water passes over the width of this kind of structure as a thin laminar sheet of supercritical flow. A fish, to swim upstream, must swim like mad through a thin film of water for the entire width of the causeway.

When the cross fall is against the flow, i.e. the high point is on the downstream side, the flow across the structure is generally deeper subcritical flow with only a short section of supercritical flow at the highest point. In this scenario, a fish can negotiate the same structure with a short burst of speed through the critical section and be in deeper, slow-flowing water just over the lip. For this project, the downstream beam was raised 100 mm over the top of the old causeway slab (see Plan 2, Stage 1 in Figure 10). The beam created subcritical flow over the entire deck of old concrete causeway.

Similarly, pipes are not generally considered good fish passage. Flow in a standard pipe configuration is all supercritical where a head of water is placed over the inlet and the downstream is flowing freely. In this project, this was overcome by incorporating the existing pipe into the fish ladder. The downstream outlet was bounded by the wall of the first step in the fish ladder to the level of the beam. This meant that all flow within the existing pipe remained subcritical. The short length of the pipe meant there was light, so it could easily be negotiated by fish making their way in either direction. Two separate sources have already reported seeing small fish and an eel negotiating the fish ladder.

Taking a fish ladder from concept to design and then to construction can be quite difficult. The concepts include sculpting the bases of the ponds for self-cleansing, minimising hydraulic jumps between the ponds (60 mm) and creating sections of laminar flow between the rocks at these jumps. For something that is really free-form concrete sculpting, it is a very difficult design to conceptualise and draw. Fortunately, Carey Molloy from MTCE went above and beyond to make his design a success, attending the site, adjusting to the site conditions and assisting with the actual construction, monitoring flow after the construction and even going back to adjust the structure once flow had settled through the structure.

Though there were some initial reservations about the design, Fisheries NSW recognised the preservation of the upstream habitat, design principles of the fish ladder and provided a permit for construction of the structure (Figure 11).

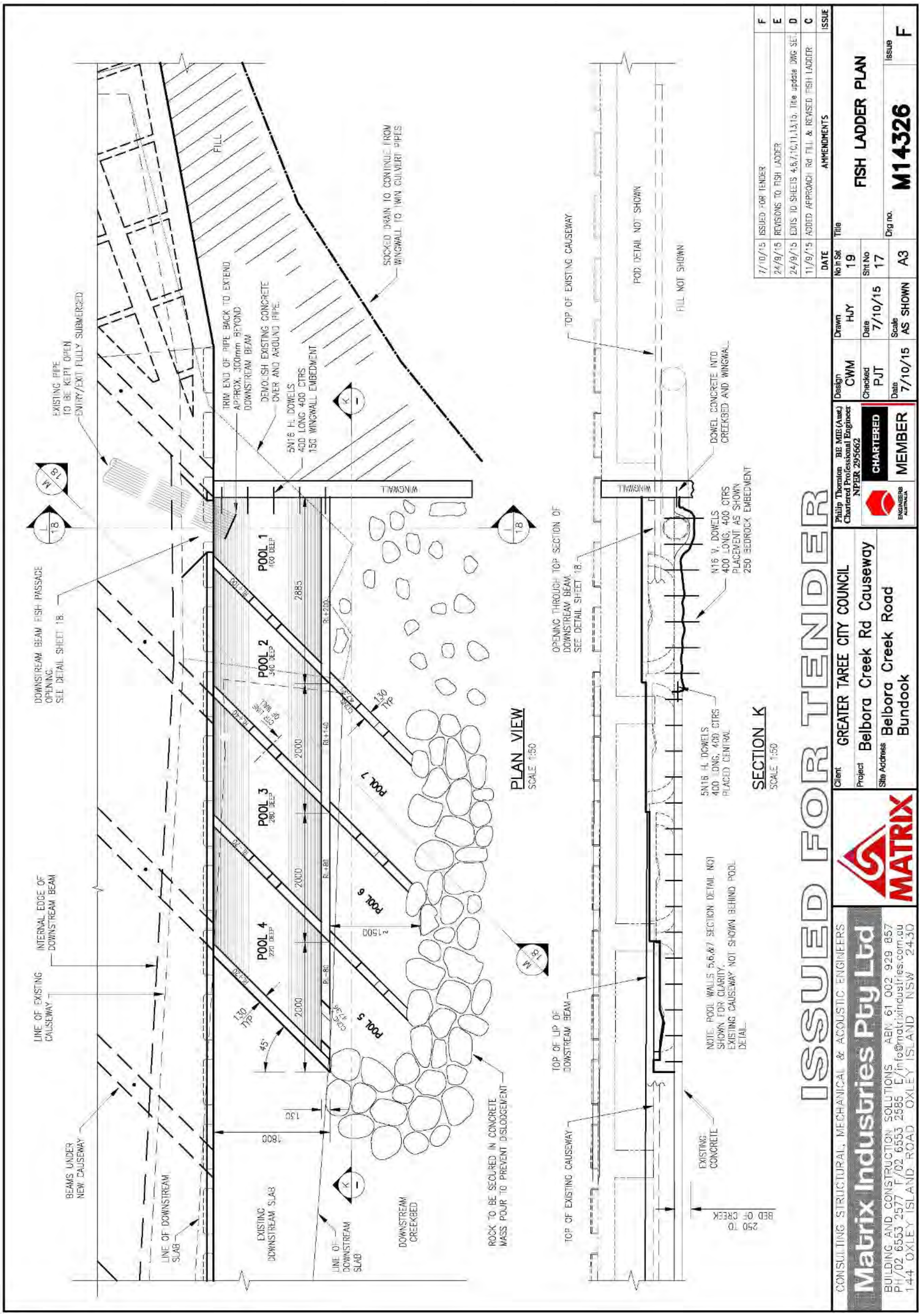

Figure 11: Plan of fish ladder and sections through upper level pools 1 to 4.

The site conditions, once the downstream headwall was removed, led to an on-site design change of the fish ladder. As can be seen in Figures 12 & 13, Pool 1 on the right back drops into Pool 2 right foreground and Pool 3 in the middle foreground before flowing back toward the causeway into Pools 4, 5 and 6 to the back left of the photographs and eventually flowing into the downstream environs in the lower left of the photographs.

Figure 12: Fish ladder in high-flow scenario.

Figure 13: Fish ladder in low-flow scenario.

3.4 The Contract

An open tender went out and some selected local concreters were alerted to the tendering process. As may be expected of a new-design construction system, the tendered prices varied wildly reflecting the unknown nature of working in a waterway on an old existing structure using an untried construction system. Local building firm Reece Construction won the tender and carried out the work in a professional manner. The project was completed successfully and without mishap in 8 weeks with a road closure period of only 3 weeks.

Continuity of access was raised as a concern to residents of the road. Council arranged with the adjoining land owner to provide local residents with secure (camera monitored) parking on the southern (Bucketts Way) side of the causeway. The contractors maintained pedestrian access across the work site at all times. This enabled the residents to park vehicles on either side of the causeway during construction and avoid the long detour around the site.

4 CONCLUDING REMARKS

The waffle pod system outlined in this paper provided a quick, cost-effective method to install a low-level bridge or culvert over the existing concrete causeway. This method maintained and built upon the existing infrastructure, provided a higher level of service to the road and in this instance preserved significant aquatic habitat and irrigation access upstream of the structure. Local tradesmen were comfortable with the construction method which did not require specialist or large lifting equipment. Thoughtful and clever design in consultation with Council officers, MTCE engineers and Fisheries NSW overcame and fulfilled the fish passage requirements of the structure. The design and construction methodology may be a useful tool for other Councils dealing with similar rural causeways in the future and looking for a cost effective solution.

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A New Method for Validating Mobile Laser Scanning (MLS) Corridor Surveys

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ABSTRACT

The introduction of Mobile Laser Scanning (MLS) in 2009 changed the approach to corridor surveys. The method has become popular as the data is collected quickly, accurately and safely. In addition, a huge swath of data is collected, enabling the whole corridor to be measured including the complete road surface. The nature of MLS lends itself to a range of road survey applications, including engineering survey for widenings, asset capture, road subsidence monitoring and clash detection. Since the introduction of MLS to the survey market, a couple of different approaches for data collection and processing have emerged. The Multi-Target (MT) approach was developed by scanner manufacturers and has been adopted by many road agencies around the world. In 2009, McMullen Nolan Group (MNG) introduced the Multi-Pass (MP) approach, which is based on survey principles and addresses some of the inherent disadvantages of Multi-Target. This paper uses two recent project examples to demonstrate some advantages of using Multi-Pass MLS. The data presented demonstrate the ability for MP MLS to be used to identify any issues with the target control coordinates, validate MLS road corridor models and detect small changes in a road corridor for the purpose of identifying areas of subsidence or change. The paper then discusses the implications of these findings for both MLS corridor model validation surveys and subsidence monitoring surveys.

KEYWORDS: *Survey model validation, multi-pass MLS, laser scanning.*

1 INTRODUCTION

Mobile Laser Scanning (MLS) was first introduced in Australia in 2009. All aspects of the technology (field collection, scanners, data processing) continue to develop and improve rapidly. Scanning is now applied to many corridor surveys and is leading the way into areas such as high-accuracy asset management and Building Information Modelling (BIM).

Over the years, two different approaches have been developed in the market for processing of MLS data along corridors:

- The Multi-Target (MT) approach was developed by many of the scanner manufacturers, which required the scanned data to be 'draped' over a rigorous survey control network. This method has become the standard for MLS surveys and has been adopted by many road authorities around the world (Caltrans, 2011; Olsen et al., 2013).

- The Multi-Pass (MP) approach was developed by McMullen Nolan Group (MNG). This approach consists of a data collection methodology as well as a processing strategy. It relies on the inherent strength of collecting redundant data to both identify outliers and improve the accuracy of the survey. The MP approach incorporates a 'self-checking accuracy system' that identifies any weaknesses in any of the survey components, whether they arise from the control coordinates and targets locations or from the scan itself (Eckels and Nolan, 2013).

MNG has been promoting the use of the MP approach for many years. We have talked about the 'theoretical' advantages of the approach – without being able to demonstrate using field data. Recently, MNG has been involved in two surveys that have enabled us to compare the MT approach to the MP approach based on real data.

This paper discusses the datasets and demonstrates how:
- MP MLS can identify errors in a control survey (that incorporates survey control and targets).
- MP MLS can find errors the original scanning survey (that were not identified using current model validation approaches).

The paper provides a summary overview of the MT and MP approaches to MLS field methodology and processing. It then investigates the two different datasets and discusses the results that were achieved. The implications of these findings on MLS road corridor surveys are then discussed in regards to:
- Control requirements – checking existing control.
- Target separation.
- Subsidence or monitoring surveys.
- MLS validation surveys.

2 REVIEW OF MLS APPROACHES

MLS is a complicated measurement process that involves a combination of scanning and imagery sensors that are mounted on a moving platform. The orientation and relative position of all the measurement sensors must be known, and the trajectory of the moving platform has to be determined to 'survey' accuracy (i.e. ±15 mm).

Many of the error sources can be minimised, measured or eliminated through an equipment calibration process (Glennie, 2007). After calibration is complete, the largest error sources that affect the accuracy of the point cloud are the positioning errors of the moving platform. The moving platform (whether it be a car, hi-rail vehicle, aircraft or boat) is positioned using both Global Navigation Satellite System (GNSS) observations and an Inertial Measurement Unit (IMU). GNSS positioning takes the form of a kinematic solution based on dual-frequency carrier phase measurements. GNSS base stations are established along the route to enable a carrier phase solution (resolved ambiguities) to be calculated each second (1 Hz). The attitude of the platform (pitch, roll and yaw) as well as its trajectory is provided by the IMU at 200 Hz, which directly measures bumps in the road and the platform position when satellite blockages occur (e.g. caused by bridges).

The errors that affect the position of the platform are related to standard GNSS measurements and comprise multipath effects and errors associated with changes in satellite configuration

(satellites rising and setting). The magnitude of the errors from satellite sources can easily be ±3 cm (Figure 1), which is higher than required for engineering surveys (Schön and Dilßner, 2007).

Figure 1: Satellite errors affecting kinematic GNSS.

A key challenge for all MLS users is to minimise or eliminate these positioning errors. To this end, two different approaches have been developed. These are known as the Multi-Target (MT) approach and the Multi-Pass (MP) approach.

2.1 Multi-Target Approach

The Multi-Target (MT) approach assumes that the trajectory of the platform cannot be determined accurately from the kinematic GNSS measurements and therefore needs to be monitored (Caltrans, 2011). Using MT, multiple targets are established along the corridor. Comparing the 'surveyed' coordinates of these targets with the measured coordinates (from the scan) enables the operator to monitor the drift and position of the scanner at regular intervals. The scan data can then be corrected and adjusted by 'pinning' it to the established control. The scan data is completely dependent on the control survey and can be imagined to be 'draped' over it.

MT requires that multiple targets be established along the corridor. These targets need to be incorporated in a survey control network that is connected to the surrounding, previously established control (Figure 2). It is recognised that the closer the target spacing, the more accurate the expected result (Soininen, 2012).

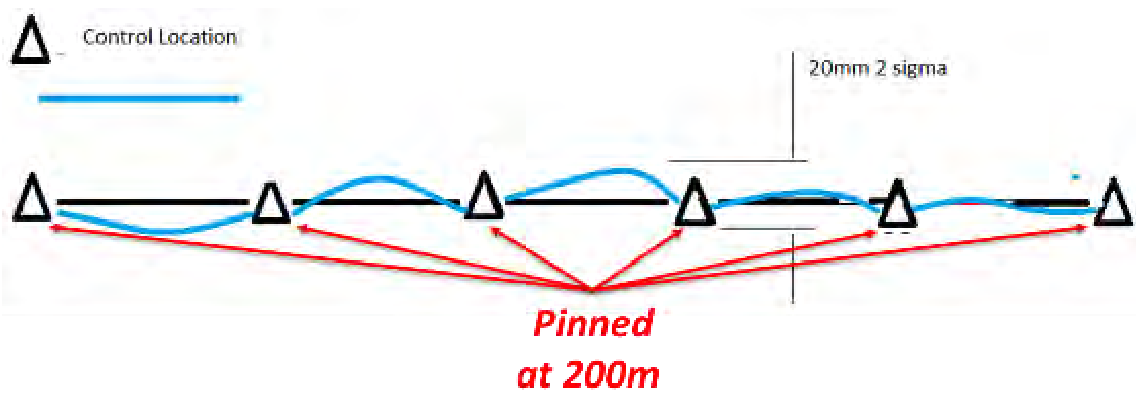

Figure 2: Multi-target approach to MLS.

One disadvantage of the MT approach is that surveyors are required to work along the corridor shoulder to establish the targets. This process often requires night work and closed lanes to ensure the safety of the survey party. Establishing targets can be time-consuming and significantly add to the cost of the job.

However, there are also some inherent dangers in the MT approach that can be very difficult to eliminate:
- Any errors in the control survey target coordinates will translate directly into the final scan surface.
- Satellite positioning errors can still occur between the targets.

2.2 Multi-Pass Approach

The Multi-Pass (MP) approach to MLS scanning creates a survey-accurate trajectory from independent measurements along a corridor. This trajectory is used to create the scan model of the corridor. Target points are required to monitor the accuracy of the scanned model and to transform the data into the local reference system.

The key to this approach is the determination of an 'averaged' trajectory (Figure 3). Each measurement epoch consists of both the GNSS trajectory (subject to satellite errors) and the scan data itself. When one measurement is compared to another, it is assumed that any differences in the results are due to satellite errors – as the road surface itself would not have moved (Eckels and Nolan, 2013).

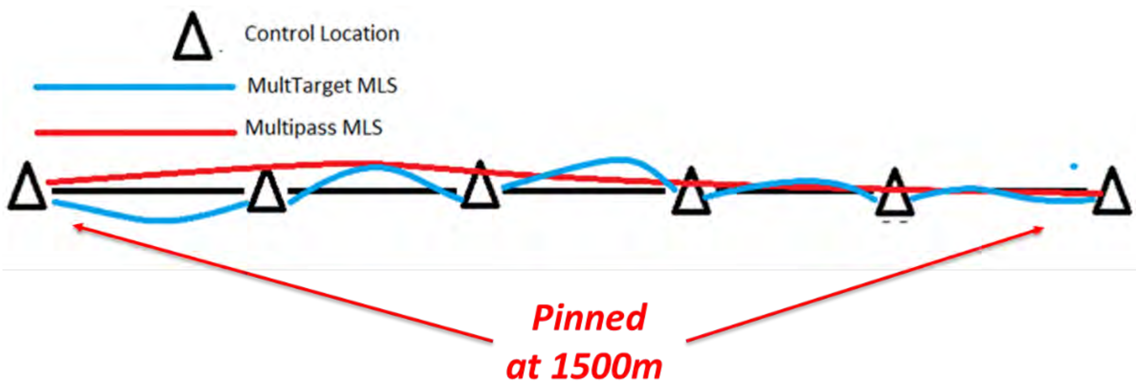

Figure 3: Multi-pass approach compared to multi-target approach.

Using MP, each corridor is measured multiple times. Each of these measurements is subject to a different set of satellite errors. The individual measurements can be plotted on a graph, which provides immediate feedback as to the quality of the data:
- If all trajectory lines are smooth and close together, the data quality is good (open road – few obstructions).
- If the trajectory lines vary greatly and separate, the data quality may be poor (when GNSS coverage is bad – GNSS obstructed corridors, tunnels, etc.).

Inspection of the plotted graph allows easy identification of outliers (bad GNSS runs), which can be omitted from further processing. The strength of MP results from combining and averaging all the runs, as the accuracy of the combined measurements is increased. The 'internal' accuracy (without considering control) can be determined, and has been found to be ±15 mm, which is an excellent result for data derived from kinematic GNSS.

The advantages of this approach are:

- It provides a 'survey-grade' measurement of the trajectory of the averaged scan. This can be compared to the coordinates of the targets (to identify targets).
- Large positioning errors caused by multipath and changes in satellite configuration are minimised as they are averaged out over multiple independent measurements.

MNG developed MP when it entered the market in 2009 and has been promoting its advantages since. From our own internal data, we have seen the errors that may occur from using single MLS runs. Recently, however, we have had an opportunity to compare some MP data to previously collected MT data and inspect the differences. We have found some interesting results, which are described in the following section.

3 DATASETS FOR REVIEW

In 2015, MNG participated in MLS surveys where MT MLS data was available. This was the first opportunity to compare the results of a MT and MP MLS survey. This section outlines the review of two such surveys performed in Australia and the United States.

3.1 Australian Dataset

The Australian dataset involved a test to monitor possible subsidence in a motorway around Sydney. The aim of the survey was to show how MP can be used to provide regular, ongoing monitoring of a road corridor to identify and quantify any movement in the road surface. This application highlights one powerful application of MLS technology.

The first dataset was collected using MT:

- A control survey was undertaken to validate the survey control coordinates around the motorway.
- Targets were placed every 150 m on both sides of the carriageway. Every second target was used to 'pin' the MLS data to the control, while the alternative targets were used in the validation survey.
- MLS data was collected and processed – this formed 'epoch 0' of the subsidence survey.

Approximately 6 months later, MNG surveyed the same section using MP:

- 16 passes of data were collected.
- Every 10th target was used (every 1,500 m) to 'pin' the survey.
- Data was processed and compared to the original survey.

Data analysis comprised two tests, i.e. the analysis of target coordinates and the comparison of the point clouds.

3.1.1 Analysis of Target Coordinates

As already mentioned, we used every 10th target for the MP approach, spaced at approximately 1,500 m. The first task was to compare the coordinates of all the intermediate targets to the coordinates generated from the MP scan. The results are shown in Figure 4. The maximum difference was 12 mm and the standard deviation 5 mm. It should be noted that these differences include errors in both the original traditional 'control' survey and in the MP survey.

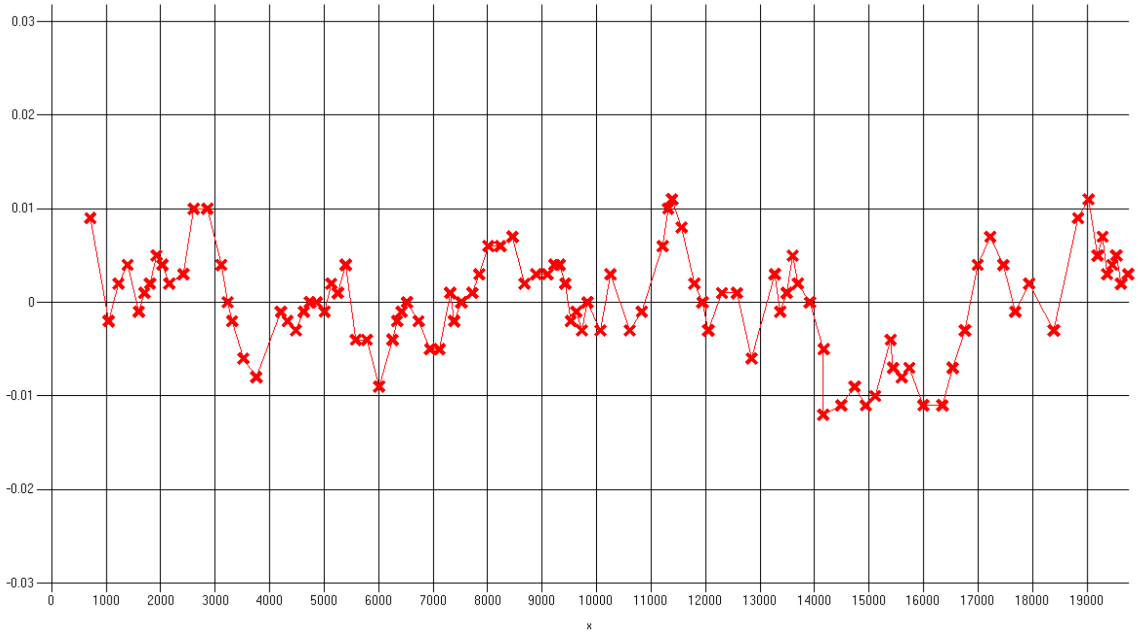

Figure 4: Target coordinates compared with MP values (values in metres).

These results indicate that the MP survey is equivalent to the accuracy of the survey control. Figure 4 also indicates that the control coordinates of all the targets were within expected tolerance.

3.1.2 Comparison of the Point Clouds

In order to compare the accuracy of the point clouds, surface points were extracted every 10 m along the centreline of each of the point clouds and compared. The maximum difference between these extracted points was 31 mm, with a standard deviation of 7 mm. This larger error appears to be due to some aberrations between some control points. Let us examine two of these aberrations.

<u>Data Comparison 1</u>

In this example, it appears that there is a significant deviation between the scanned surfaces (Figure 5). An image of this section is provided in Figure 6.

After some discussion, it was determined that this section of the motorway had been re-surfaced after the initial scan. The MP scan was able to measure the change in height of the road section and the extent of the re-surfacing.

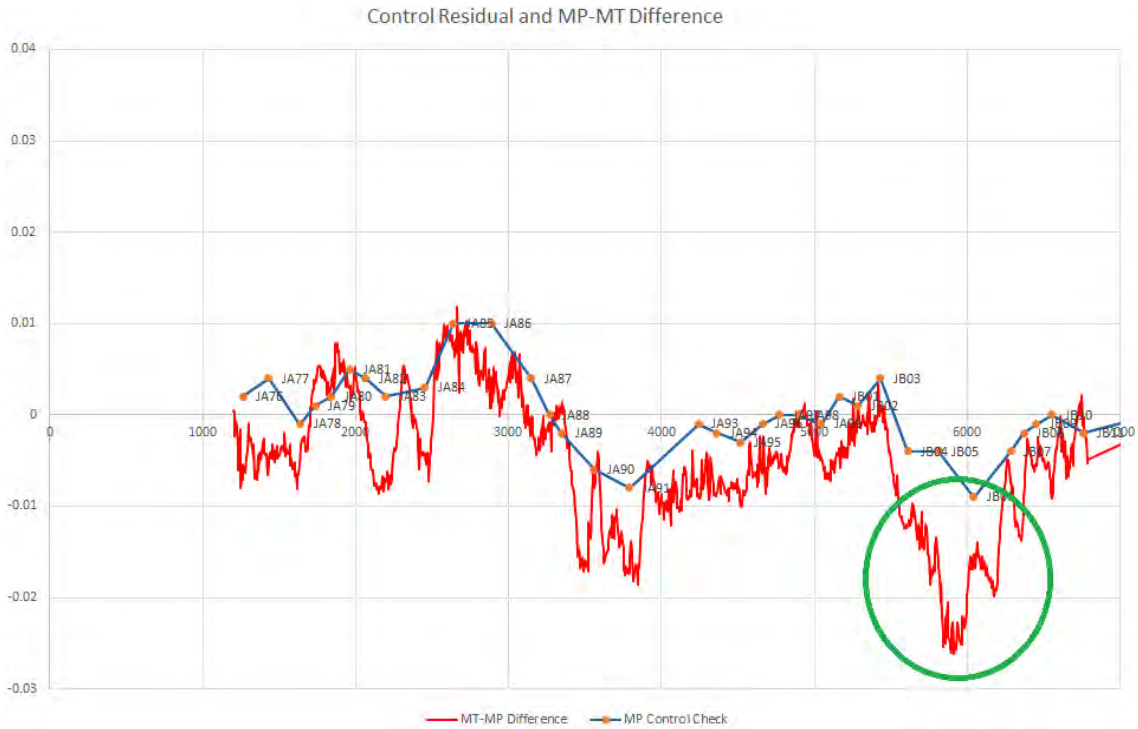

Figure 5: Difference in point cloud 1 (values in metres).

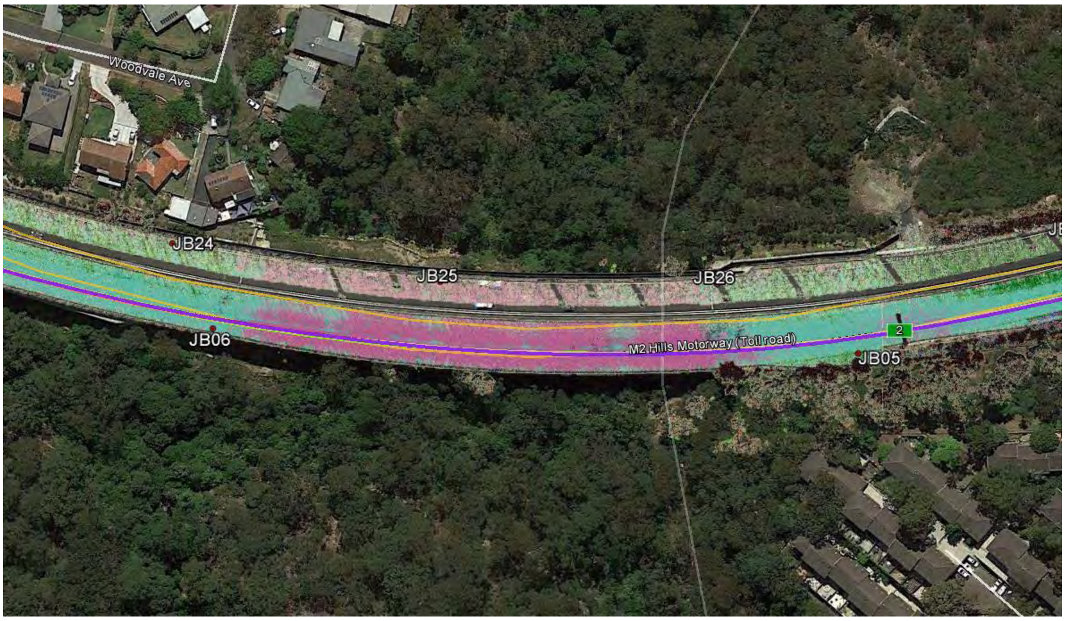

Figure 6: Image of point cloud deviation 1.

Data Comparison 2

In this example, we investigate the large deviation in the top right of Figure 7. An image of the deviation is provided in Figure 8.

This survey indicates a variable deviation between two control points. The point clouds agree at each of the target points, but diverge in the middle of the points to 31 mm. This indicates that a satellite error may have occurred between the control points in the trajectory calculation

of the initial scan (Figure 9). This example shows how MP scanning can be used and applied to both verification surveys and monitoring or subsidence surveys.

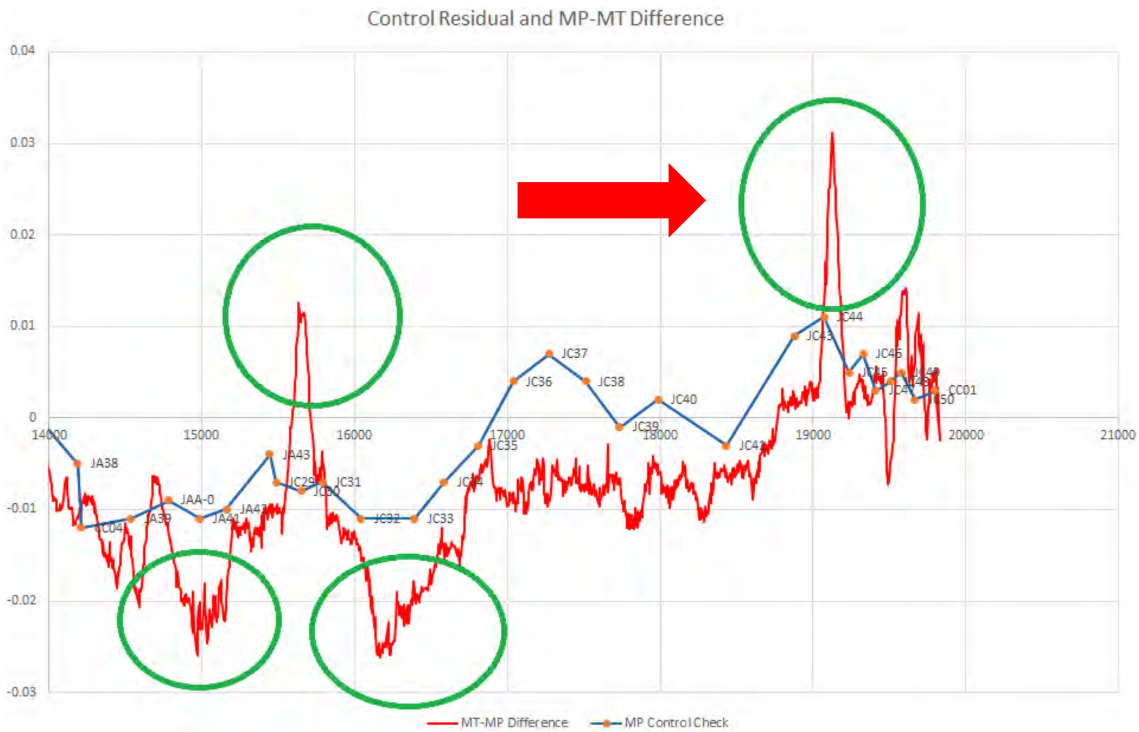

Figure 7: Difference in point cloud 2 (values in metres).

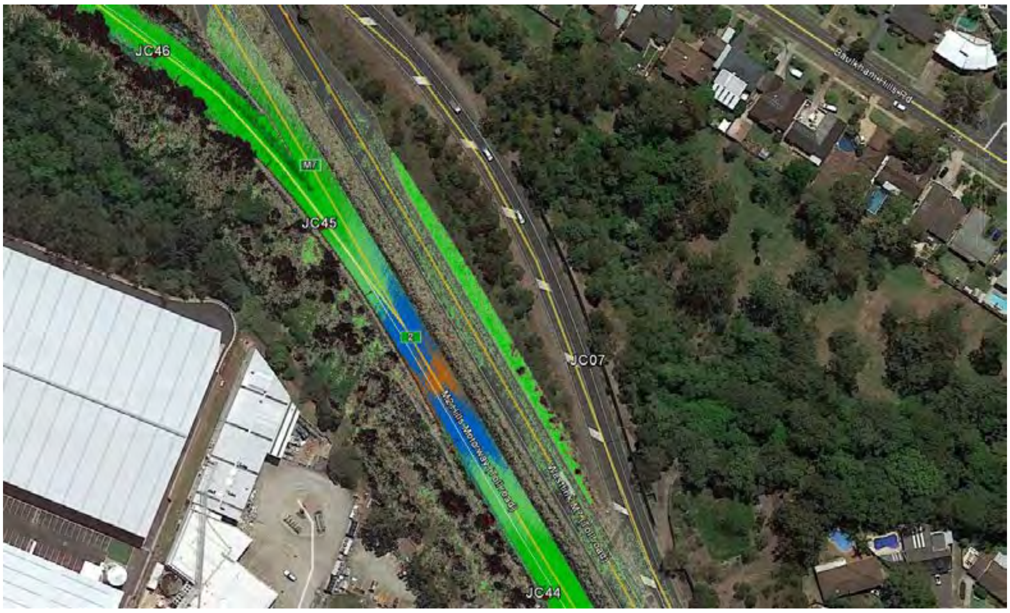

Figure 8: Image of point cloud deviation 2.

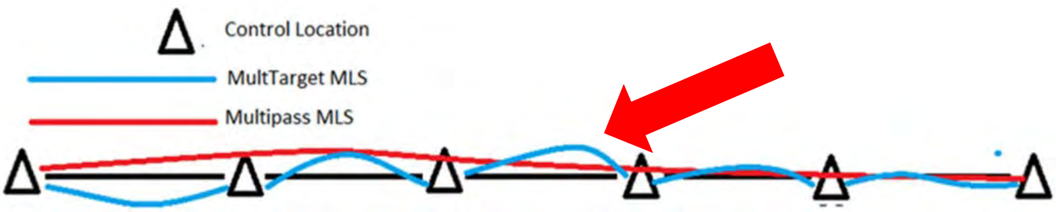

Figure 9: Possible cause of point cloud deviation.

3.2 U.S. Dataset

MNG was invited to participate in a MP MLS survey along 20 km of a motorway in Florida in 2015 (Figure 10). The contractor we worked with was required to provide MLS data urgently and had completed a MT survey of the site. The aim of our survey was to demonstrate the advantages of MP by collecting data along the same stretch of freeway. The MT survey had checked all local control and placed targets along each side of the corridor. MNG collected the MP data and carried out a similar analysis to that done in section 3.1.

Figure 10: Motorway in Florida.

3.2.1 Analysis of Target Coordinates

The first task undertaken was to check the control coordinates of the targets with those supplied by the contractor. It was found that one of the target control points disagreed with the measured height by 78 mm (Figure 11).

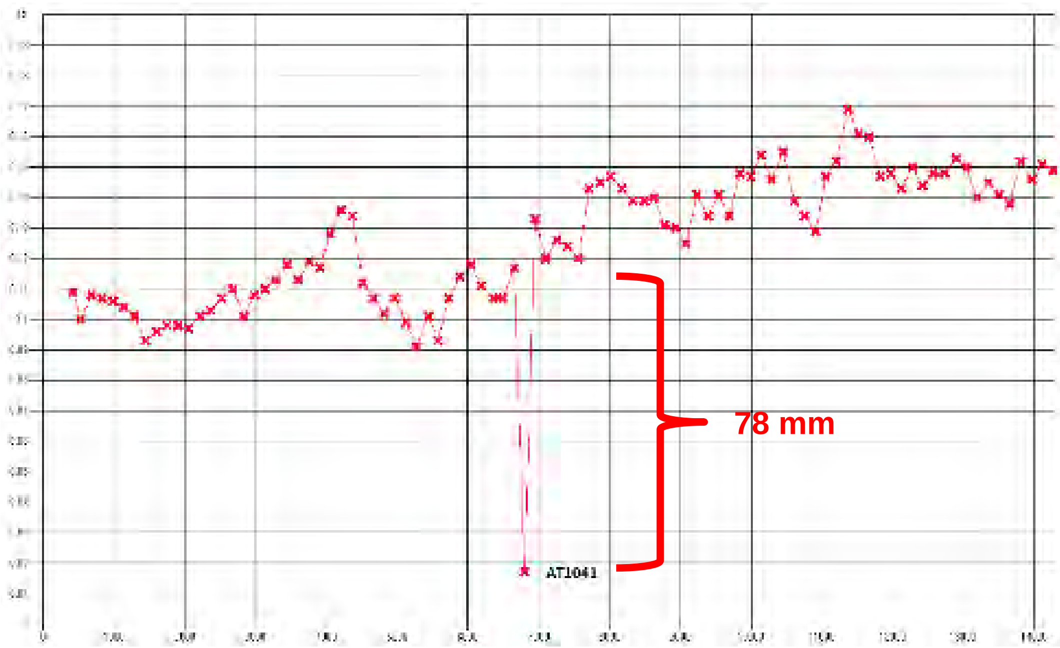

Figure 11: Comparison of control coordinates vs. MP MLS, showing observed spike in control at AT1041.

The discrepancy occurred at point AT1041 (eastbound carriageway) but did not affect AT1150 on the westbound carriageway (Figure 12).

Figure 12: Control coordinate discrepancy.

What happened to this survey? Under the stress of time constraints and contractual obligations, we found that the data had already been delivered to the customer. The coordinates of AT1041 were not checked in the field.

It seems the MLS data had been processed as follows:

- The MLS data from each carriageway was processed separately, using the appropriate targets.
- When transforming the MLS dataset from its 'standard' WGS84 (GNSS) reference frame to the North American Datum, neither control point AT1041 nor AT1150 were used as a common point.

When comparing the MT point cloud (delivered) to the MP point cloud (validation survey), we can see how this 78 mm was distributed (Figure 13):

- On the eastbound carriageway, the point cloud model was 19 mm above the control mark.
- At the bridge overpass (crossing the freeway), there is a 44 mm discrepancy between the eastbound and westbound carriageway where the point clouds met.
- On the westbound carriageway, the point cloud model was 15 mm below the mark.
- The 78 mm error had been spread across the bridge overpass.

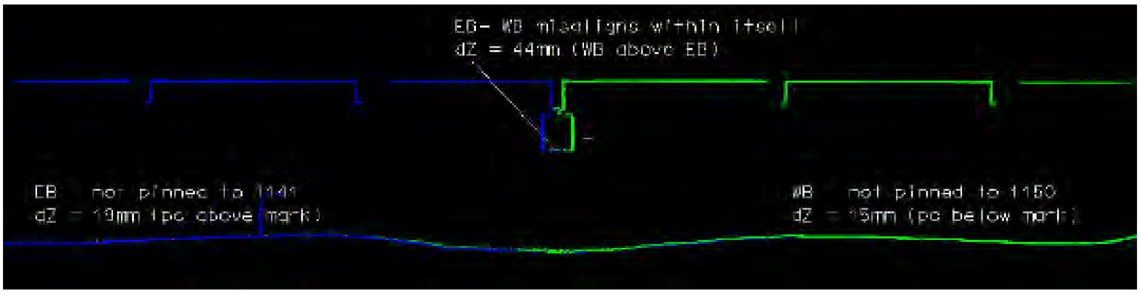

Figure 13: Cross section of overpass at discrepancy.

This dataset demonstrates the power of MP MLS to check control and validate surveys. It also highlights the errors that can be introduced to survey measurement if correct model validation is not completed.

4 DISCUSSION OF RESULTS

The datasets investigated in section 3 demonstrate the power of the MP approach as both a data collection and data validation method:

- Combining independent datasets to create an 'averaged' trajectory (control polyline) produces a 'survey-accurate' measurement that can be used to check control.
- The MP process shows that the 'jumps' present in scanning data caused by satellite errors (multipath and satellite configuration changes) are smoothed out. These jumps can occur at any time and can affect MT data, regardless of spacing between targets.
- MP data collection provides an ideal tool for data validation surveys, and also subsidence and monitoring surveys. Small changes in the road surface can be identified in extent and magnitude.

But what are the consequences of these findings? How do they affect what we currently survey and what implications do they have for other surveys in the future?

4.1 Validation of MP Approach

The survey data presented in this paper is further evidence that MP MLS provides 'survey-accurate' MLS data, independent of ground control. The MP approach also provides benefits in safety, cost and time. The data shows that:

- Less control is required on a MP MLS survey to achieve the required accuracy.
- Any errors in control can be identified from the MP MLS scan.

It is important to note that survey control and corridor targets are still required to check the accuracy of the scan and to provide common points for data transformation into the local reference system. Targets are particularly relevant in areas of poor GNSS coverage (i.e. urban canyons, tree-lined streets etc.) where the trajectory of the scanner is reliant on the IMU.

4.2 MP for MLS Validation Surveys

All MLS road corridor surveys require model validation. Current quality assurance procedures employ traditional survey techniques to 'validate' a road model at intervals along the corridor. While this approach is appropriate for traditional corridor surveys, it may be sub-optimal for validating MLS surveys. Some of the concerns are:

- Only a very small percentage of the road model is tested.
- Requires surveyors to be on the road to take the measurements.
- Relies on the integrity of the control survey.

The data presented in this paper suggests that a better alternative for validating road surface models is to use a second independent multi-pass MLS survey of the road. The multi-pass survey is independent as it uses different control targets than the original survey and is observed under different satellite configuration conditions. The advantages of using this approach are:

- 100% of the model is validated – not just a small subset.
- Errors in the control survey can be identified.
- All survey work can be completed from the safety of the scanning vehicle.

4.3 MP for Subsidence or Monitoring Surveys

The datasets indicate that MP is well suited to high-accuracy monitoring surveys, as the approach is suited to measuring both the extent and magnitude of any hardstand surface changes in the road corridor. These changes may be planned (e.g. road re-surfacing), or unplanned (e.g. subsidence).

MLS surveys for subsidence monitoring conducted on a regular basis can quickly identify any changes in the road corridor. Tools have been developed to allow the customer to easily visualise the difference in surface levels from one measurement epoch to another, allowing the identification of 'suspect' movement, where further, localised monitoring surveys can be undertaken.

5 CONCLUDING REMARKS

MNG has been involved in scanning since 2009. We have developed the multi-pass approach for MLS with its associated data collection methodology and data processing. MNG has always believed that the MP approach provides real benefits to our customers, in regards to high-quality and high-accuracy data. The datasets presented in this paper provide direct evidence of these benefits. The datasets also indicate that MP MLS provides an excellent platform for road corridor validation surveys and subsidence and monitoring surveys.

ACKNOWLEDGEMENTS

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Archival Recording of a Historically Significant Site Using 3D Laser Scanning Technologies

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ABSTRACT

The Windsor Bridge Replacement project required the survey and modelling of the State Heritage listed Thompson Square Conservation Area, existing Windsor Bridge and immediate surrounds using 3D laser scanning. This is one of the first times where a project's Condition of Approval has required archival recording to be completed using 3D laser scanning technology. An archival model of the project area was produced using a combination of an existing G73 compliant site survey, Terrestrial Laser Scanning (TLS), Mobile Laser Scanning (MLS) and multi-beam sonar bathymetry. The point clouds from the various surveys were combined into a single unified, spatially accurate, full-colour point cloud. 3D string lines were extracted for heritage building facades, the top and underside of Windsor Bridge, along with other streetscape detail. The resulting 3D point cloud and model was used to derive a range of deliverables, including detailed ortho-rectified elevations of the street frontages showing far more detail and colour than traditional CAD drawings, current views of the site that match location and perspective views of historic photographs enabling more recent features to be removed to reveal how the site could look if these features were removed, interactive panoramic images allowing virtual visits to the site from the desktop, and a number of video 'fly-throughs' that highlight the historical significance and issues associated with the site and construction of the new replacement bridge. Proposed design and changes could be dropped into the point cloud model to assess any potential issues early. The level of detail and accuracy captured by the 3D scanning process has set a new benchmark in the archival recording process for what is considered one of the oldest public squares in Australia. This archival recording will be put to use in the long-term strategic planning for conserving the heritage fabric within the township of Windsor.

KEYWORDS: *Laser scanning, TLS, MLS, archival recording, heritage.*

1 INTRODUCTION

Roads and Maritime Services are undertaking a project to replace the existing road bridge over the Hawkesbury River at Windsor with a new bridge located 35 metres downstream. Originally built for horse-drawn vehicles and foot traffic in 1874, Windsor Bridge is now used by up to 19,000 vehicles a day, which has contributed to the deterioration of the structure over time which no longer meets current engineering and road safety standards.

The alignment of the new bridge, selected during the option assessment phase, lies within the eastern approach of Thompson Square Conservation Area (TSCA). This precinct has a significant number of heritage-listed buildings of local and state significance in addition to the existing bridge, which is also a heritage-listed structure (Heritage Council of NSW, 2008). Due to this sensitivity of existing structures, appropriate management measures including 3D archival recording were required to be implemented to ensure appropriate recording has taken place and safeguards are in place prior to construction activities.

With the complexities of the existing constraints of the project, Jacobs demonstrated to Roads and Maritime the application and benefits of 3D laser scanning technology, not only for the 3D archival recording process but also for design and engineering purposes. While Jacobs was specifically only engaged for completion of the 3D archival recording, Jacobs have been able to utilise the spatial data for more definitive engineering analysis during progression of the detail design. The benefits of being able to interrogate the detail design with the comprehensive spatial dataset has enabled Jacobs engineers to visualise and assess with greater certainty the associated impact of the project’s footprint against the existing heritage features of the site.

2 PROJECT BACKGROUND AND REQUIREMENT FOR ARCHIVAL RECORDING

In 2013, as part of the Conditions of Approval (CoA) for the project, *a detailed survey and analysis of the Thompson Square Conservation Area, Windsor Bridge and the immediate surrounds using 3D laser scanning* was set by the Director General of the then NSW Department of Planning and Infrastructure. The 3D archival recording, to be completed prior to commencement of pre-construction works, required capturing all the historic heritage sites within the Strategic Conservation Management Plan (SCMP) study area, existing Windsor Bridge and immediate surrounds highlighted in Figure 1. In addition, the project’s selected archaeologist was required to complete standard photographic and archival recording of all affected heritage sites as identified in the Environmental Impact Statement for the entire project site inclusive of the SCMP study area.

The CoA detailed the 3D archival recording process is to be undertaken in accordance with the guidelines issued by the NSW Heritage Office (1998). However, this guideline does not include guidance specifically for 3D archival recording. In order to overcome this initial project challenge, Jacobs and Roads and Maritime worked closely together to develop a coordinated site plan to undertake the 3D survey so all captured data including heritage structures, existing road structures, bathymetry of the existing river and immediate surrounds would integrate seamlessly as one package.

This approach enabled multiple deliverables to be produced from the one dataset including the required archival recording but also a complete 3D survey model, which has assisted with design constraint reviews and progressing the detailed design for the project.

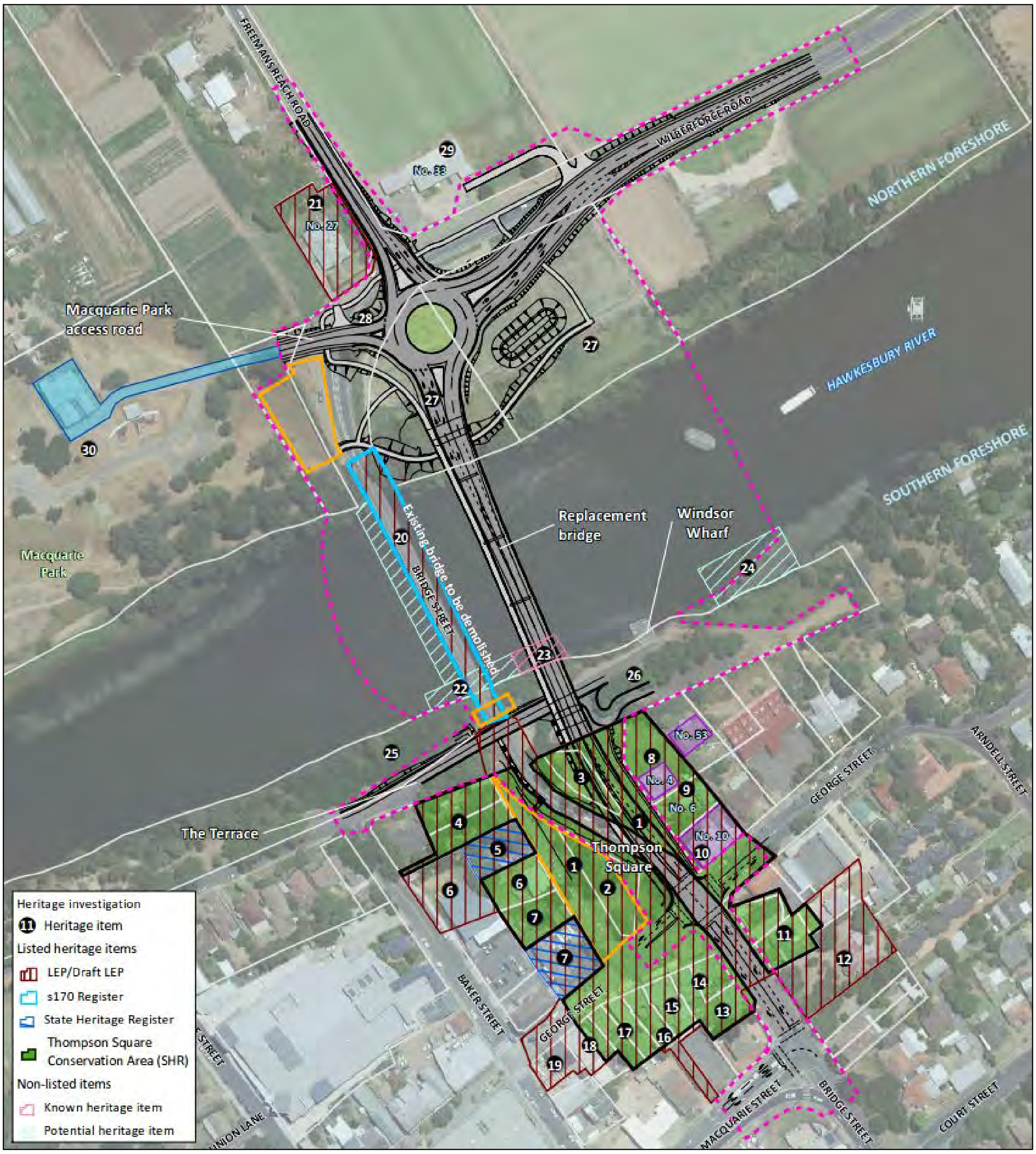

Figure 1: Heritage investigation units.

3 LOCATION

The project is located at Windsor in the Hawkesbury local government area about 57 km north-west of Sydney. The township of Windsor is located on the south bank of the Hawkesbury River at the foot of the Blue Mountains. The proposed bridge works and associated road works that make up the project extend from the intersection of Freemans Reach Road and Wilberforce Road in the north to the intersection of Macquarie Street and Bridge Street in the south. The extent of Jacobs's 3D scanning survey is shown in Figure 2.

Figure 2: Extent of survey.

4 METHODOLOGY

A multi-purpose model of the project area was produced using a combination of an existing G73 compliant site survey, Terrestrial Laser Scanning (TLS), Mobile Laser Scanning (MLS) and multi-beam sonar bathymetry. The point clouds from the various surveys were combined into a single unified, spatially accurate, full-colour point cloud. 3D string lines were extracted for heritage building facades, the top and underside of Windsor Bridge, along with other streetscape detail.

4.1 Existing G73 Roads and Maritime Survey

As with all Roads and Maritime projects, a state-based datum was used. In this case, the Map Grid of Australia 1994 (MGA94) and the Australian Height Datum 1971 (AHD71) were adopted. For this project, Roads and Maritime had already carried out a G73 compliant feature survey of the site, which is detailed in Figure 3. This existing survey was quality checked by Jacobs surveyors and used, along with a number of marks on public record in the Survey Control Information Management System (SCIMS – see Kinlyside, 2013), to help control and verify our 3D laser scan surveys.

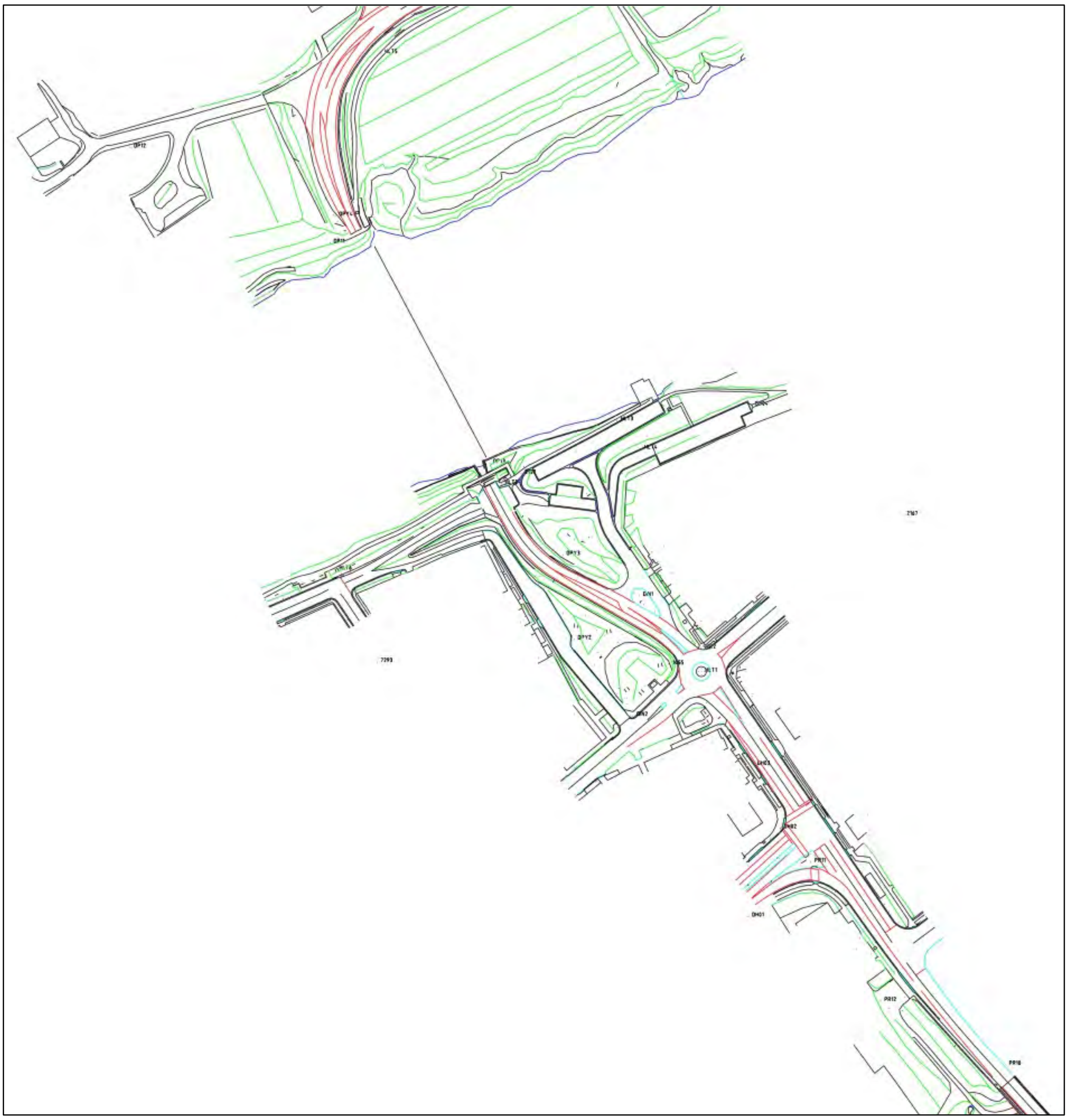

Figure 3: Existing G73 compliant survey provided by Roads and Maritime.

4.2 Terrestrial Laser Scanning

Terrestrial Laser Scanning (TLS) was carried out using a Leica HDS7000 scanner and a Pentax DSLR camera fitted with a fisheye lens. Static scans were acquired from a number of positions across the site from locations optimised to capture the multi-faceted heritage buildings, bridge abutments, Thompson Square Park and features unlikely to be visible from the roads or river. Capture was carried out without entering private property or interrupting traffic flow.

Using the existing Roads and Maritime survey control network, a Leica total station was used to coordinate targets visible in the scans as per general arrangement in Figure 4. These coordinated targets were later used to position and orientate each of the static scans onto the project datum. This is Jacobs' preferred method of TLS control in an outdoor and elongated project. This method minimises the propagation of errors that occurs using point cloud to point cloud registration.

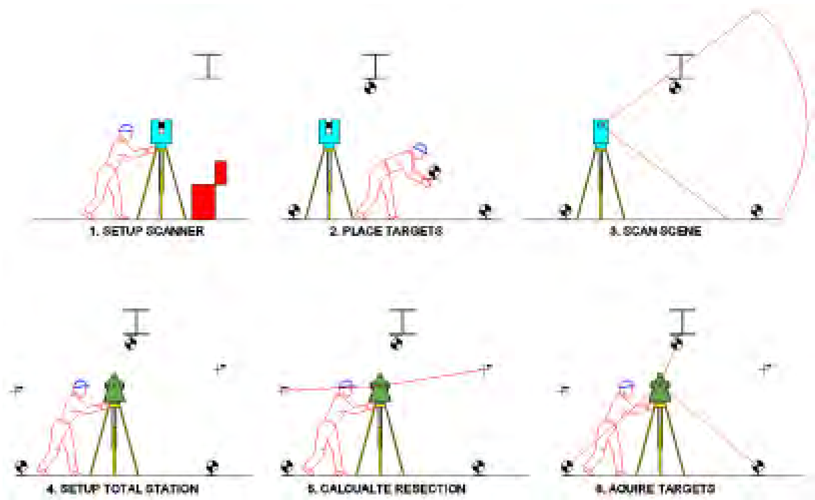

Figure 4: Static scanning process.

Photography was taken at each scan site using a Nodal Ninja tripod mount that allows the camera and the scanner to share a common focal point. Eight photographs were taken at each scan site, which were later stitched together to produce a high-resolution 360º panoramic image showing what was visible from each scan site (Figure 5). The panoramic images were later precisely referenced to the scans and used to colourise the point cloud, allocating each point in the point cloud its own unique colour value.

Figure 5: Panoramic image from TLS.

4.3 Mobile Laser Scanning (Land)

A Riegl VMX-450 Mobile Laser Scanner (MLS), fitted with a LadyBug 5 Panoramic camera system and mounted on a vehicle was used to acquire point cloud data from all trafficable areas throughout the site (Figure 6). The MLS point cloud was used to capture the road pavement, verify and/or update existing Roads and Maritime detail and complete the capture of the heritage building facades where it was difficult for the TLS to get line of sight.

Figure 6: Riegl VMX450 MLS system.

All trafficable roads within the site were driven between 2-4 times in each direction to provide redundancy, ensure maximum coverage and assist in maximising spatial accuracy. Capturing multiple passes helps eliminate anomalies caused by Global Navigation Satellite System (GNSS) or Inertial Measurement Unit (IMU) noise/error and eliminates data gaps caused by moving objects obscuring line of sight.

The GNSS and IMU trajectory data from the MLS system along with the GNSS base station data was post-processed to improve the resulting MLS point cloud. The calculated deviation vectors between multiple overlapping point clouds (from multiple passes) along with the trajectory quality file are used to adjust the trajectory and the resulting laser data to produce a single unified point cloud.

Distinct features from the original Roads and Maritime feature survey and the terrestrial laser scan point cloud were then selected and used as control to further refine the MLS point cloud to within better than ±15 mm of the original Roads and Maritime survey provided. Figure 7 provides a snapshot of this process. It was found that a few sections of kerbs and line work had changed since the original Roads and Maritime survey. These features were updated using the MLS point cloud.

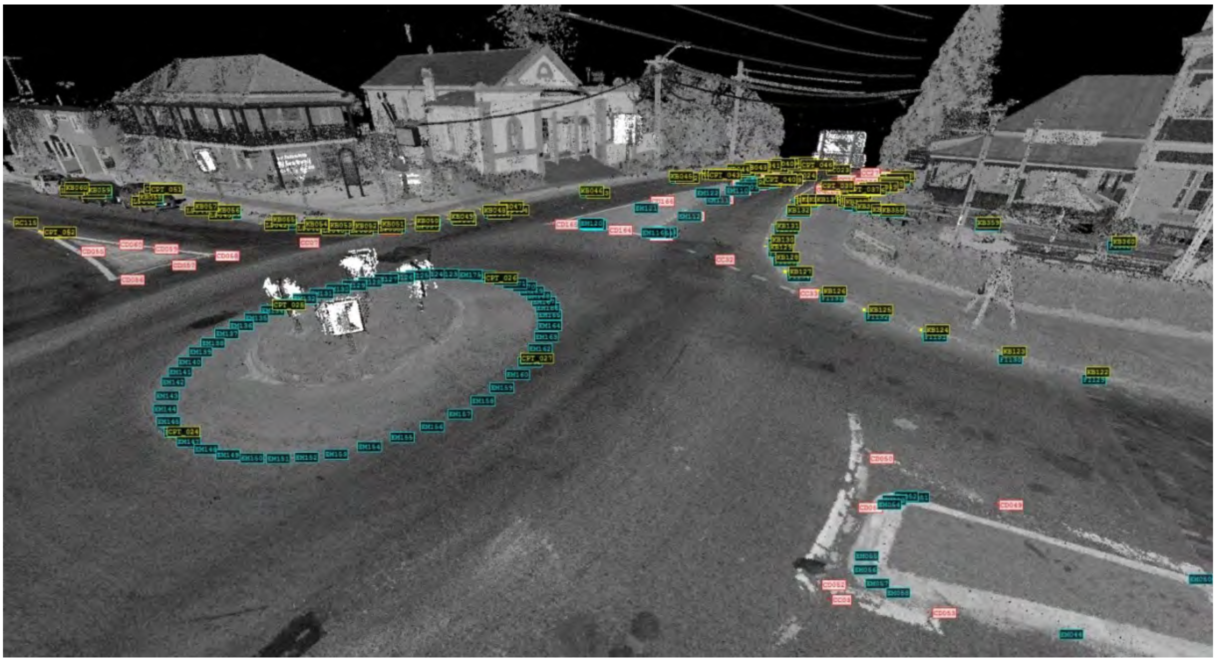

Figure 7: Example control points used to refine the position of the MLS point cloud.

The LadyBug camera is integrated into the Riegl VMX-450's system. The panoramic images from this camera are georeferenced with the laser data and then used to colourise the point cloud, allocating each point in the point cloud its own unique colour value (Figures 8 & 9).

Figure 8: Georeferenced images and point cloud from MLS.

Figure 9: Combined colourised point cloud from MLS and TLS.

4.4 Mobile Laser Scanning (Water) and Bathymetry

The river bed, adjacent banks and the underside of the historic Windsor Bridge were all surveyed from a small survey vessel operated by Astute Surveying (Figure 10).

Figure 10: Maritime survey vessel.

For surveying the river bed, the vessel was fitted with a Multi-Beam Echo Sounder (MBES) and a range of other sensors, including Motion Reference Unit (MRU), Differential Global Positioning System (DGPS) gyro or heading sensor, Real Time Kinematic (RTK) surveying system, and Sound Velocity Probe (SVP) for determining the velocity of sound within the water column.

For surveying the river banks and the underside of each span and the support structures of the historic Windsor Bridge, the vessel was also fitted with Jacobs' Riegl VMX-250 MLS system with its own dedicated GNSS and IMU systems.

Initially the MLS point cloud was processed to an estimated accuracy of ±40 mm using the GNSS and IMU data recorded during acquisition. The point cloud was further adjusted to match the overlapping multiple passes and the controlled MLS and TLS point cloud of the bridge structure and abutments captured from above. There was only a small amount of controlled point cloud overlap and some structural assumptions were required to fit the point cloud into position, as highlighted in Figure 11.

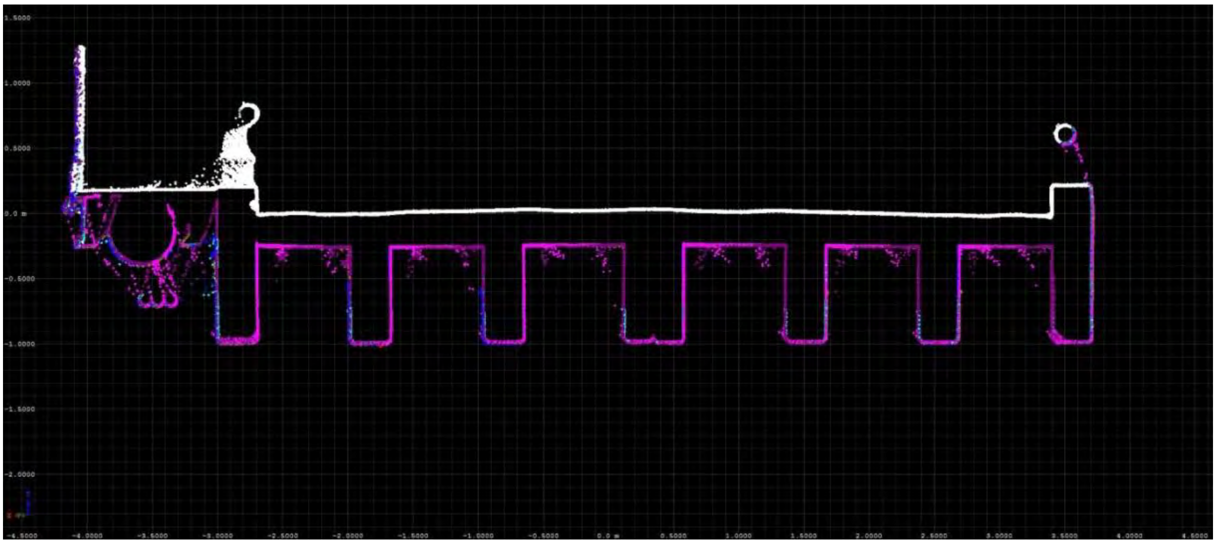

Figure 11: Adjustment of point cloud collected from vessel (purple) to point cloud collected from vehicle (white).

4.5 Combining the Various Surveys

Other than ensuring that all the individual surveys were accurately controlled and in the same reference frame, there were a number of other tasks required to bring all the datasets together into a single model. Figure 12 details the different surveys undertaken across the project area.

The intensity ranges for scanned points varies between scan systems. As a result, there was a requirement to adjust the intensity ranges so that the intensity values for points on similar objects were consistent, regardless of which of the three scan systems was used (Figure 13).

Classification of the points within the point cloud is helpful to assist in the extraction of 3D features and makes interpretation of the data more efficient. In this instance, to assist in modelling the area, the combined point cloud was classified into 'ground' and 'non-ground' classes using an automated approach. A manual cleaning process was then undertaken to remove noise and points on vehicles, people and other mobile objects picked up during the scanning phase.

Figure 12: TLS positions (red), road MLS trajectory (green) and water MLS trajectory (blue).

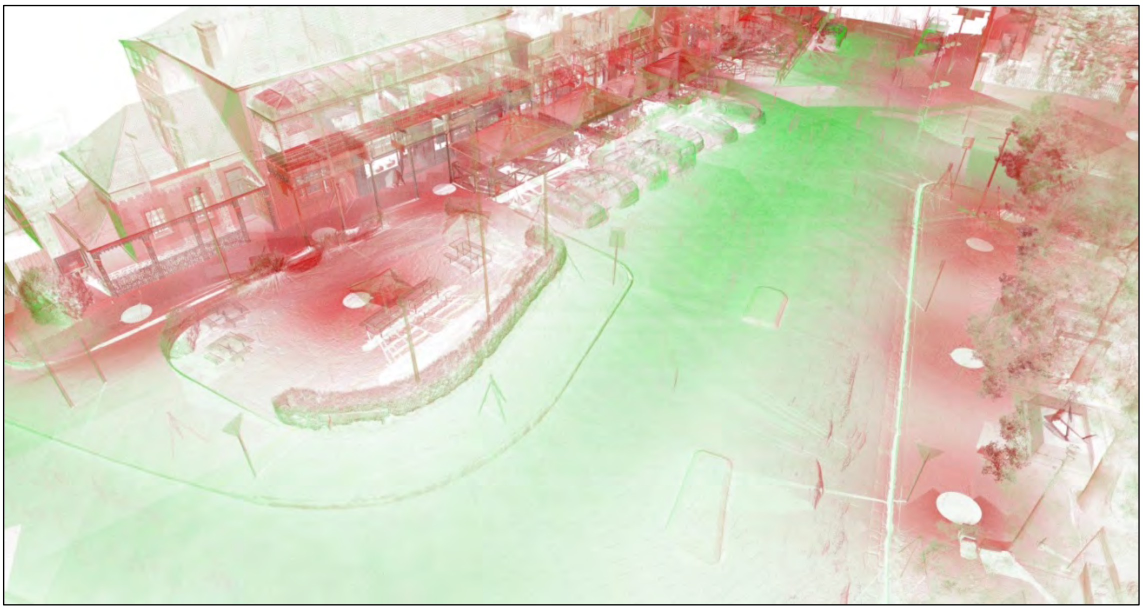

Figure 13: Combining TLS points (red) and MLS points (green).

4.6 Modelling of the Heritage Facades, Windsor Bridge and Streetscape

Using the combined, data-rich point cloud, 3D line strings of the main elements of the heritage buildings were extracted in 3D Computer-Aided Design (CAD). The detailed and colour point cloud was used to supplement the line strings and provide the finer detail and context.

A full 3D model of the historic Windsor Bridge including pylons, flanges, steel beams, services, road pavement, footpath, guardrails and fences was also extracted from the unified point cloud. In addition, the survey model provided by Roads and Maritime was further enhanced to include additional streetscape furniture and detail.

5 RESULTS

The final data-rich 3D survey model contains both point cloud and 3D line strings derived from traditional field survey, three separate scan systems and a multi-beam echo sounder. From this model a number of project deliverables have been produced to date, including those described in this section.

5.1 Combined Georeferenced Point Cloud (With Colour)

The combined point cloud provides maximum coverage and detail of all areas within the scope of the project. This point cloud is the base data for a snapshot-in-time record of the Windsor area. This data can be revisited time and time again to extract additional information for new purposes.

The dense coloured point cloud has the advantage of being both photo-realistic and spatially accurate, but is also measureable, and can be visually manipulated based on the characteristics of the laser return and post-processing (e.g. point source, intensity, classification, height, laser pulse deviation, pulse return number, RGB value, etc.).

The point cloud was converted to a number of formats to enable it to be referenced and used in a range of software platforms utilised by each of the engineering disciplines involved in the project.

5.2 Georeferenced Images

Position and orientation information was produced for the panoramic images. These images were imported into CAD software to assist in 3D modelling. Interactive panoramic images where produced for the project captured simultaneously during the 3D mobile laser scanning process. While similar to Google street view, these images provide a more up-to-date imagery dataset and ability to view completely in 360º perspectives (Figure 14). To date this has helped with initial planning of pre-construction site investigations, assisting with identification of existing constraints on site.

An example was accurately determining the existing overhead cabling and tree crowns in proximity to geotechnical investigations. By reviewing the interactive panoramic images it prompted the use of a smaller sized drilling rig more suited to the surrounding constraints, allowing works to be completed on site safely and without delays (Figure 15).

Figure 14: Panoramic images from MLS and TLS.

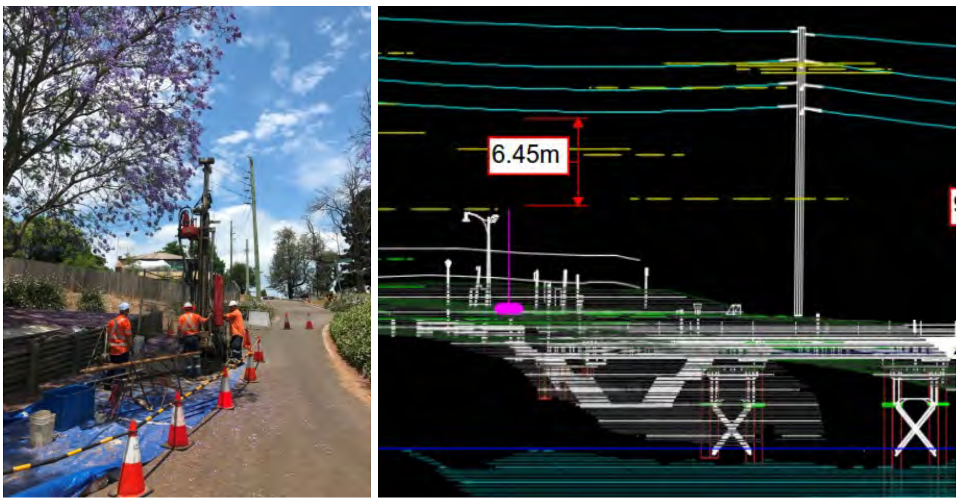

Figure 15: Geotechnical drilling works on site and survey clearance checks.

5.3 Historical Windsor Bridge: Plans, Sections and Elevations

Plans and sections, including both the extracted 3D model and point cloud, accurately detail the bridge structure above, underneath and below water. The point cloud is detailed and dense enough to show concrete condition (Figure 16).

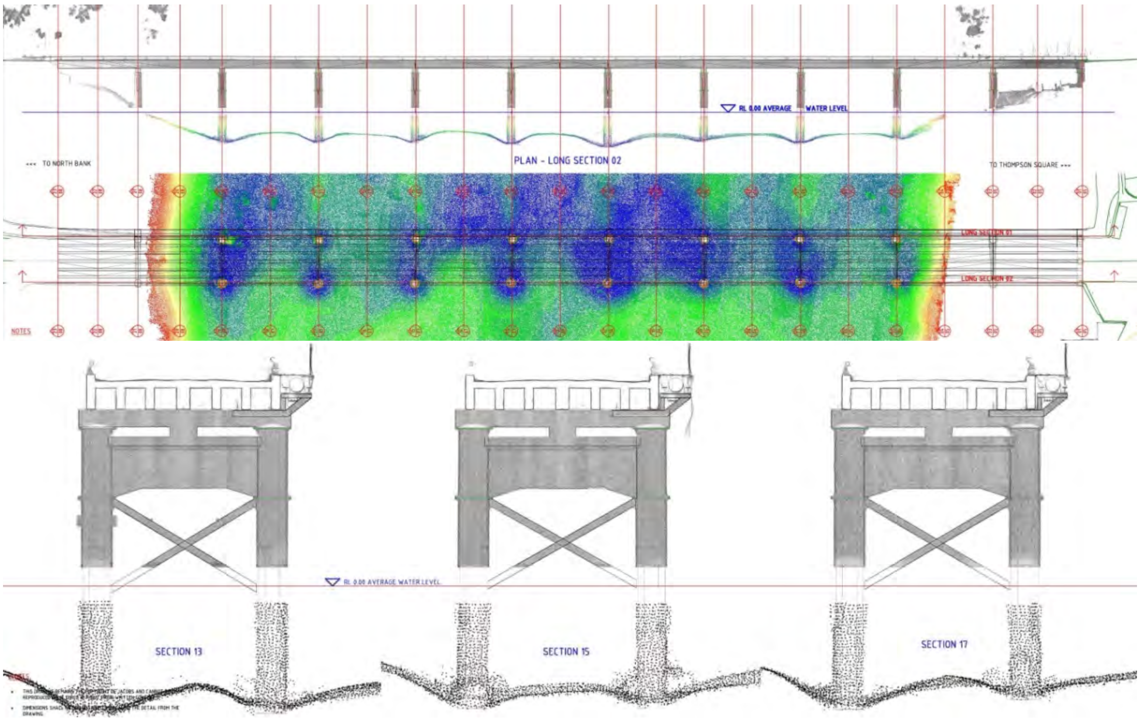

Figure 16: Plans and sections of Windsor Bridge.

5.4 Ortho-Rectified Elevations of Heritage Building Façades, Streetscapes and Thompson Square

Elevations, including point clouds, have greater detail and context than traditional CAD drawings alone. These elevations combine the spatial accuracy and historical context/detail. Due to the high density and photorealism, details as small as 10 mm have been captured and shown efficiently and accurately (Figures 17-19).

Figure 17: Street front perspective of the heritage-listed Macquarie Arms Hotel.

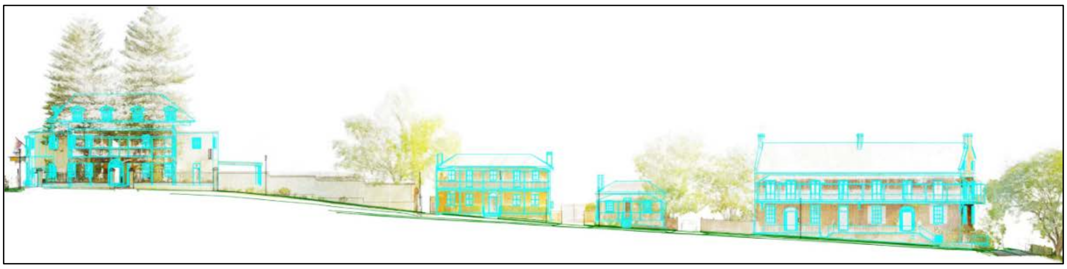

Figure 18: Streetscape of Thompson Square Road.

Figure 19: Section of Thompson Square Park.

5.5 Cross Sections and Elevations of the River Bank Including Above and Below Water

The twin scanners on Riegl VMX-250 (and 450) are capable of collecting multiple data points (targets) per laser pulse, enabling far greater penetration through dense vegetation. By filtering out all but single and last targets, the ground surface could easily be identified and extracted. This information was used by Jacobs' geotechnical engineers to plan river bank stabilisation for the proposed new bridge (Figure 20).

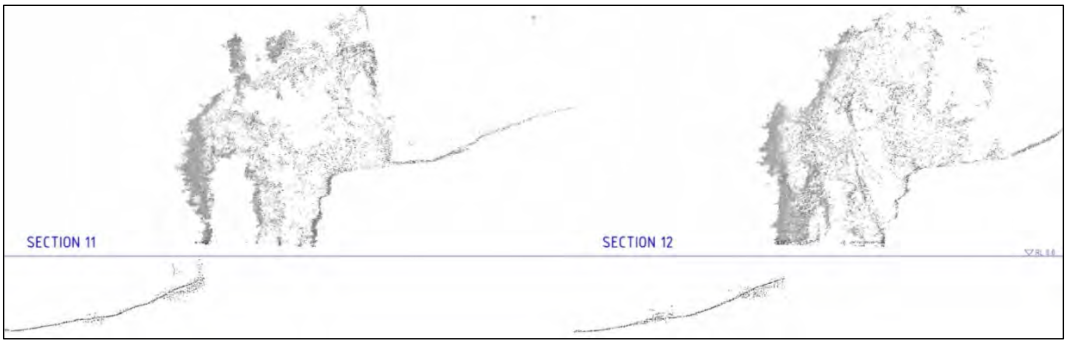

Figure 20: Point cloud cross sections of the south river bank.

5.6 Historical Photos Integrated into the 3D Point Cloud

Current views of the site that match location and perspective views of historic photographs were produced by Jacobs to investigate the visual impacts of removing more recent features, like trees, to reveal how the site might look if these features were actually removed. This has been a useful tool in the development of the urban design and landscaping plan for the project, which requires a design to be sympathetic to the historical views and heritage values of the Macquarie era township of Windsor (Figure 21).

Figure 21: Historical photos of Thompson Square overlayed with point cloud survey data.

5.7 Design and 3D Survey Overlays

During the development of detail designs for large scale infrastructure projects, conflict and constraints analysis is traditionally performed on plan in 2D. With capture of the 3D survey data, the Windsor Bridge project team has been able to go a step further and assesses proposed design changes and review clashes or constraints by dropping the design into the point cloud model (Figure 22).

This has been particularly important in identifying potential issues early on in the design phase with the ability to navigate the design through the existing urbanised environment which has significant heritage constraints present. The outcome of this design review process has given the project team greater confidence in mitigating possible impacts to heritage items during construction of the new replacement bridge and approaches.

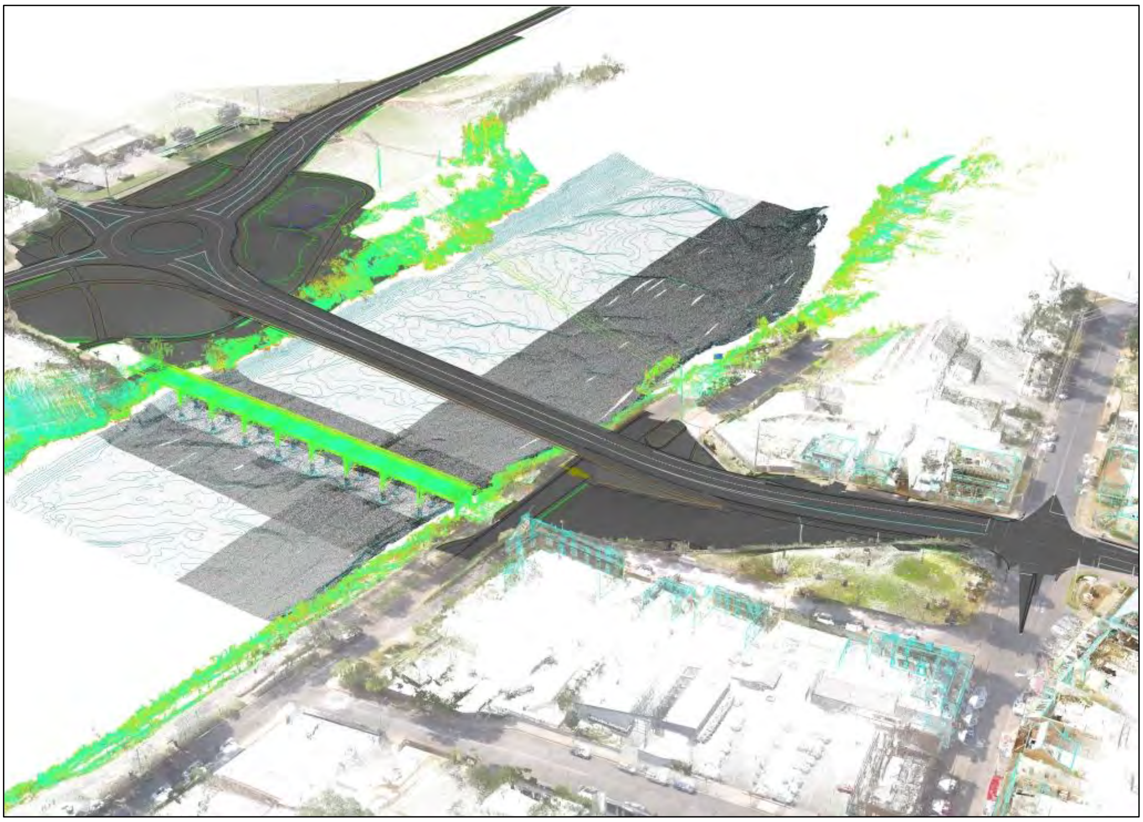

Figure 22: Point cloud data with detail design overlaid.

6 CONCLUDING REMARKS

The coordinated approach of undertaking the 3D archival survey and modelling of the State Heritage listed Thompson Square Conservation Area, existing Windsor Bridge and immediate surrounds using 3D laser scanning by Roads and Maritime and Jacobs enabled the various surveys to be combined into a single unified, spatially accurate, full-colour point cloud. This resulted in the development of a high-density, data-rich 3D model of the site which in turn has contributed to the project team developing multiple innovative survey outputs and archival recording techniques that have been received well by the Office of Environment and Heritage who are the Heritage Council of NSW's representative for the project.

The level of detail and accuracy captured by the 3D scanning process has set a new benchmark in the archival recording process for what is considered one of the oldest towns in Australia. Along with the improvements to the detail design associated with avoiding impacts to existing heritage structures, this archival recording will be put to use in the long-term strategic planning for conserving the heritage fabric within the township of Windsor.

ACKNOWLEDGEMENTS

The Roads and Maritime Survey Section and the Hawkesbury Regional Museum are gratefully acknowledged for their assistance in preparing this paper.

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Compensation for Property Improvements in the ACT: A Temporal Perspective

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ABSTRACT

The compulsory acquisition of rural land involves compensation for improvements such as buildings, fencing and pasture improvements. Should that also include timber removal completed perhaps a hundred years prior? How do you determine the number of trees that were removed and how do you calculate the value of tree removal? This paper looks at some aspects of such a case in the Australian Capital Territory from 2007. After the 2003 bushfires in the Australian Capital Territory (ACT), the ACT Government took the opportunity to start urban development in the Molonglo Valley. In addition to developing former forestry land, a number of rural properties were acquired and their leases terminated. Three of the affected rural lessees challenged the Government's combined valuation of $4 million and lodged their own claim which was about $28 million. Their claim was overwhelmingly based on the removal of trees from the three properties up to 100 years previous. A report addressing the lessees' tree removal claim was commissioned by the ACT Government in preparation for an arbitration hearing. The report looked at the historical state of trees on the three affected properties from pre-settlement to the current day. While the report covered many issues, this paper focuses on three aspects of the report. These include the effect of cold air drainage on the establishment of trees, the use of old portion plans to estimate historical tree densities and the use of mapping and aerial photography over the last 100 years. In summary, the ACT Government report known as the Ingwersen Report demonstrated the tree densities across the three properties were probably 17 to 30 trees per hectare and would probably not have exceeded 34 trees per hectare. This was well short of the 200 to 300 trees per hectare claimed by the lessees. An arbitration hearing, in November 2007, rejected the lessees' claim for extra compensation and ruled in favour of the ACT Government with costs.

KEYWORDS: *Cold air drainage, tree densities, compensation, portion plans, aerial photography.*

1 INTRODUCTION

Three property holders being Coonan, Tanner and Tully held leases over a number of rural blocks in the Molonglo Valley, in the Australian Capital Territory (ACT). The three leases over the properties expired in 2005 (Figure 1):

- Coonan: Block 181 (north), District of Belconnen and Block 181 (south), District of Weston Creek as shown in DP 5132 and DP 5133 respectively.
- Tanner: Block 428, District of Stromlo as shown in DP 8114.
- Tully: Blocks 1171 & 1187, District of Weston Creek as shown in DP 9458.

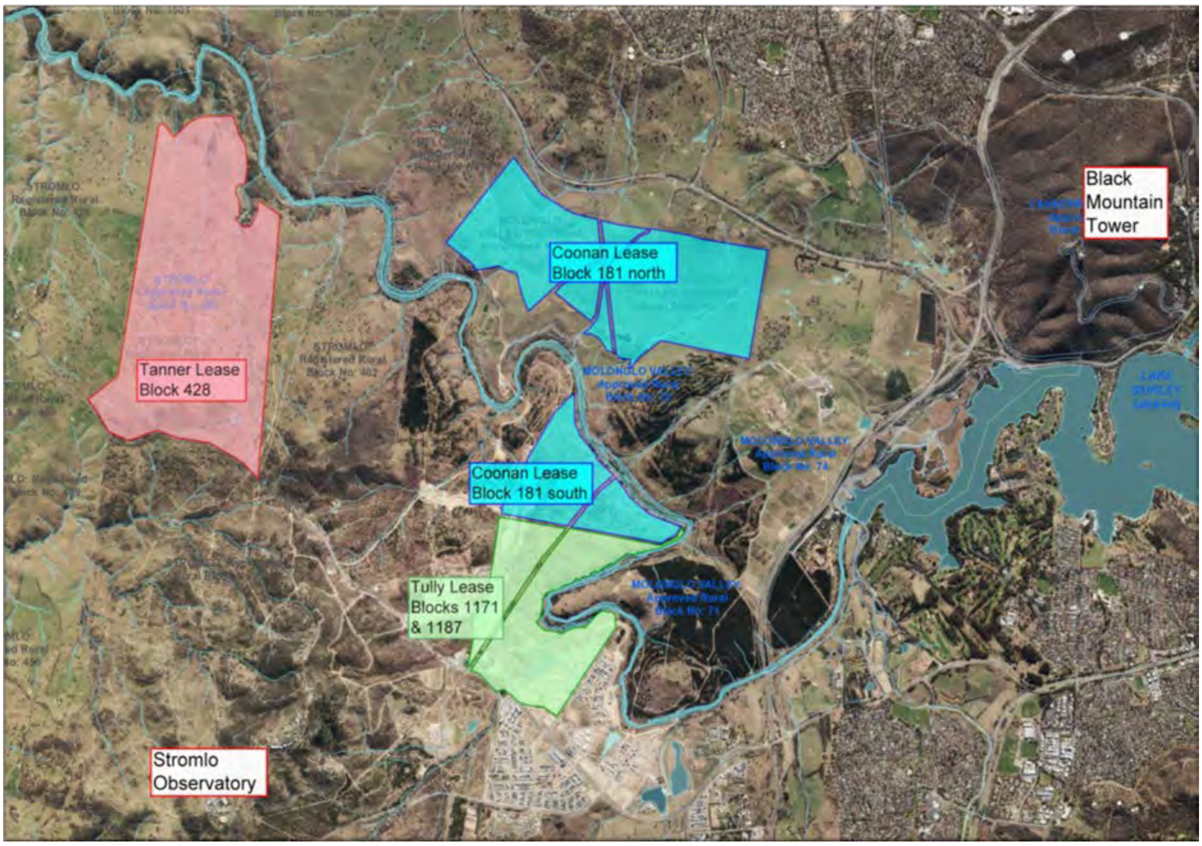

Figure 1: The location of the three properties with 2015 aerial imagery.

It should be noted that all ACT land is divided pursuant to the Districts Act 2002. All parcels are uniquely identified as follows:

- Rural parcels by Block Number and District Name.
- Urban parcels by Block Number, Section Number, Division Name and District Name.

Each of the leases included Clause 2(c) which stated the Commonwealth (*or Territory Government*) covenants with the lessee

> "*to pay to the lessee at the expiration of or sooner determination of said term the value at such expiration or determination, as the case maybe, of all fixtures and erections on, and* ***of all improvements on or affected by the lessee or by a prior lessee under this lease or under a prior lease*** *of the land comprised in the lease at such expiration or determination, except such fixtures, erections and improvements, if any, as are expressly excepted from the purchase by the lessee..., such value to be ascertained by agreement or in default of agreement by arbitration...*"

The ACT Government had valuations prepared for all lands included in the three leases totalling $4 million. The lessee's combined valuation was $28 million. The lessees claimed they were entitled to compensation for the historical timber removal improvement affected by previous lessees under Clause 2(c) of their respective leases. In order to support the claim, a vegetation report was prepared by Cumberland Ecology Pty Ltd to establish the state of the forestation of the properties in its virgin condition. The report became known as the Cumberland Report.

2 THE CUMBERLAND REPORT

The Cumberland Report (Cumberland Ecology, 2007) estimated there were trees between 20 to 30 metres high across the three properties, at the time of European settlement and at a likely density between 200 to 300 trees per hectare. Together with an independent cost analysis, the resultant timber removal value of about $24 million was determined. The Cumberland Report determined Open Forest, Grassy Woodlands and Dry Scelrophill Forest covered almost all subject properties. Grassy Woodlands was the dominant vegetation classification and the density was in excess of 200 trees per hectare (Table 1).

Table 1: Summary of the results for the three subject properties from the Cumberland Report and an estimate of the total number of trees removed assuming the remnant trees represent 10% of the original number.

Property	Area (ha)	Pre-European Vegetation Communities	%	Area (ha)	Tree Height	Tree Density (Stems/ha)	No. of Trees (Min)	No. of Trees (Max)
Coonan	437.6	Open Forest	5	21.88	> 30 m	250 - 300	5,470	6,564
		Grassy Woodlands	95	415.72	> 25 m	200 - 250	83,144	103,930
		Dry Sclerophyll Forest						
Tanner	451	Open Forest	5	22.55	> 30 m	200 - 250	4,510	5,637
		Grassy Woodlands	65	293.15	> 25 m	200 - 250	58,630	73,287
		Dry Sclerophyll Forest	30	135.30	> 20 m	250 - 300	33,825	33,825
Tully	192.3	Open Forest	5	9.62	> 30 m	200 - 250	1,924	2,405
		Grassy Woodlands	95	182.69	> 25 m	200 - 250	36,538	45,672
		Dry Sclerophyll Forest						
					90%	Clearance	198,036	244,188

3 THE INGWERSEN REPORT

In response to the lessees' claim, the ACT Government engaged Dr F. Ingwersen to prepare a report on the historical tree cover and densities across the three properties (Ingwersen, 2007). The two main aims of the Ingwersen Report were to:

1. Determine the likely vegetation classification which would have existed prior to European settlement.
2. Estimate the tree density which would have existed pre-settlement, at the time the Commonwealth Territory was established (1912) and at different years relevant to the commencement of leases over the subject properties.

The Ingwersen Report determined the most appropriate vegetation classification for the subject properties was either Woodland or Grassland. Furthermore, it claimed the subject properties had a likely tree density of about 30 trees per hectare. The Ingwersen Report claimed there were a number of reasons for the huge difference in tree densities estimated for the pre-settlement or early-settlement era. One of the main reasons being the incorrect misinterpretation of the relevant vegetation category derived from the Australian Survey and Land Information Group (AUSLIG) Atlas of Australian Resources Vegetation Map (AUSLIG, 1974).

The difference in tree densities estimated also resulted from a number of other arguments presented in the report. This paper will examine three of the main arguments presented by Ingwersen. These are:

1. The effects of cold air drainage to restrict tree growth across most of the subject properties. Within these areas Grassland was the dominant vegetation type.
2. The use of relevant New South Wales (NSW) portion plans from the period of early European settlement in the 19th century, which provide indicative notations about vegetation types and the estimation of the maximum tree density based on distance from reference trees to portion corners.
3. The use of 20th century mapping and aerial photography to identify a distinct tree line consistent with the exclusion, from pre-settlement times, of trees in the lowest parts of the subject properties and to confirm the state of the tree cover at crucial years relevant to the leases from the 1920s to 2004.

3.1 Effects of Cold Air Drainage

Ingwersen describes Cold Air Drainage (CAD) as a night-time, winter phenomenon in the highlands of south-east Australia which results in the pooling of very cold air on valley floors and hollows. The resultant frosts on low ground are enough to suppress tree establishment and significantly influence vegetation patterns.

Ingwersen cites reliable scientific studies that have noted the phenomenon in the highlands and, in particular, the ACT:

1. Taylor (1910) states "the timberline is a fairly well marked feature and roughly corresponds to the 2000 foot or 610 m contours in the neighbourhood of the capital site. Below this line the country is open and indeed almost treeless – partly natural and partly artificial".
2. Pryor (1939) made reference to the effects of severe cold on the Canberra plains as the main factor in excluding natural timber cover.
3. The Bureau of Meteorology (1968) noted an annual mean of 77 days of frost temperatures in Canberra. The number of days is variable dependent on local topography.
4. Moore and Williams (1976) noted "treeless plains occur at as low as 400 to 600 metres at Canberra" and "grasslands occupy the floors of shallow saucer-shaped valleys, the lower slopes of which are covered by sub-alpine woodlands".

Ingwersen generally adopts, as the upper limit of CAD effects, the lower margin of the mature standing surviving tree line. This local tree line is observable on modern maps and aerial photos and corresponds approximately with the 560 m contour line. Consistent with Moore and Williams (1976), there are very few trees below the 560 m limit (Figure 2).

Ingwersen pointed out the CAD limit may vary across some of the subject properties south of the Molonglo River where the north-eastern aspect of the land may lower the 560 m limit to 550 m or even 540 m. Some stands of trees below the 560 m limit on Block 428 in the District of Stromlo are Snow Gums which are suited to the lower winter temperatures.

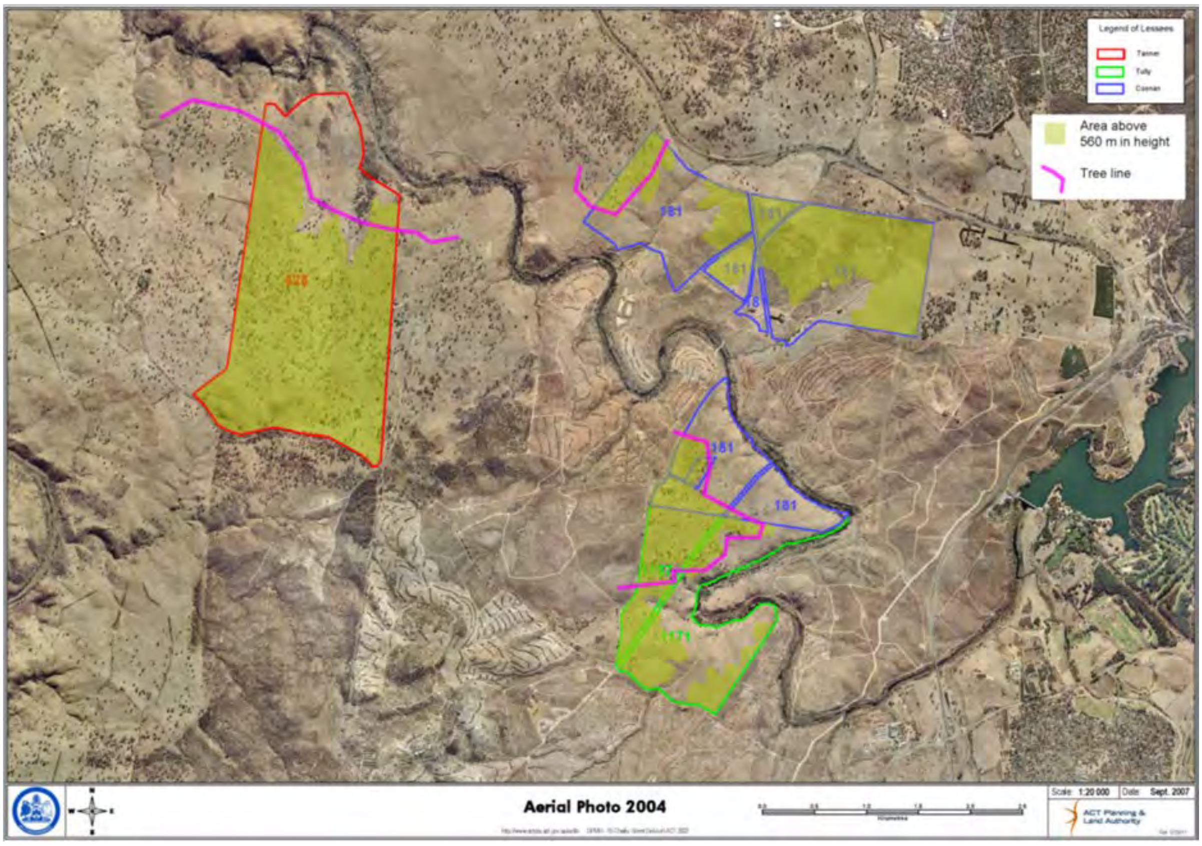

Figure 2: The 2004 aerial imagery showing the areas above 560 m in elevation and related tree line (courtesy of ACT Government Survey Office).

3.2 The 1830s to the 1910s and the Use of NSW Portion Plans

The NSW portion plans provided effective supporting evidence for Ingwersen's tree density estimation. When the Commonwealth Territory was created, the NSW portions were maintained as the basis for the Territory's rural cadastre. As time went on, Territory deposited plans gradually replaced the portion plans and leasehold title replaced freehold title. Freehold title was not completely extinguished in the Territory until the 1970s.

All the original NSW portion plans, within the Territory, were reproduced on behalf of the Commonwealth and certified as shown in Figure 3. The original as well as certified copies are currently held by the ACT Office of the Surveyor-General.

3.2.1 The Use of Portion Plan Annotations

The annotations on the portion plans gave some insight as to the vegetation cover from the original surveyor involved. Not all plans included such annotations but those that did generally described the vegetation as open forest and sometimes well grassed (Figures 3 & 4). Some notations indicated a value for the clearing and therefore indicating some clearing occurred post European settlement, but prior to survey in the 1870s and 1880s (see Figure 3).

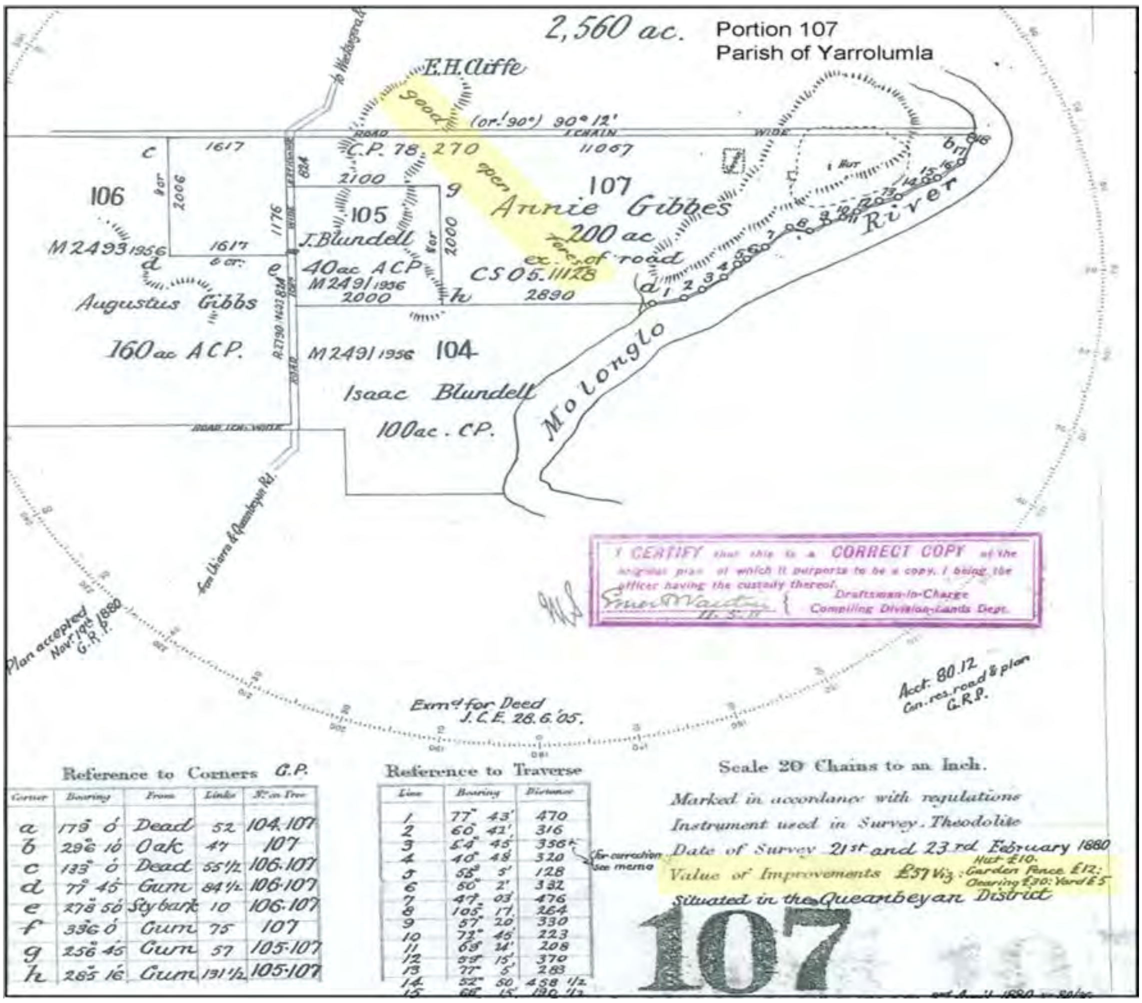

Figure 3: Part of the ACT certified plan for Portion 107 in the Parish of Yarrolumla (ACT Government Survey Office).

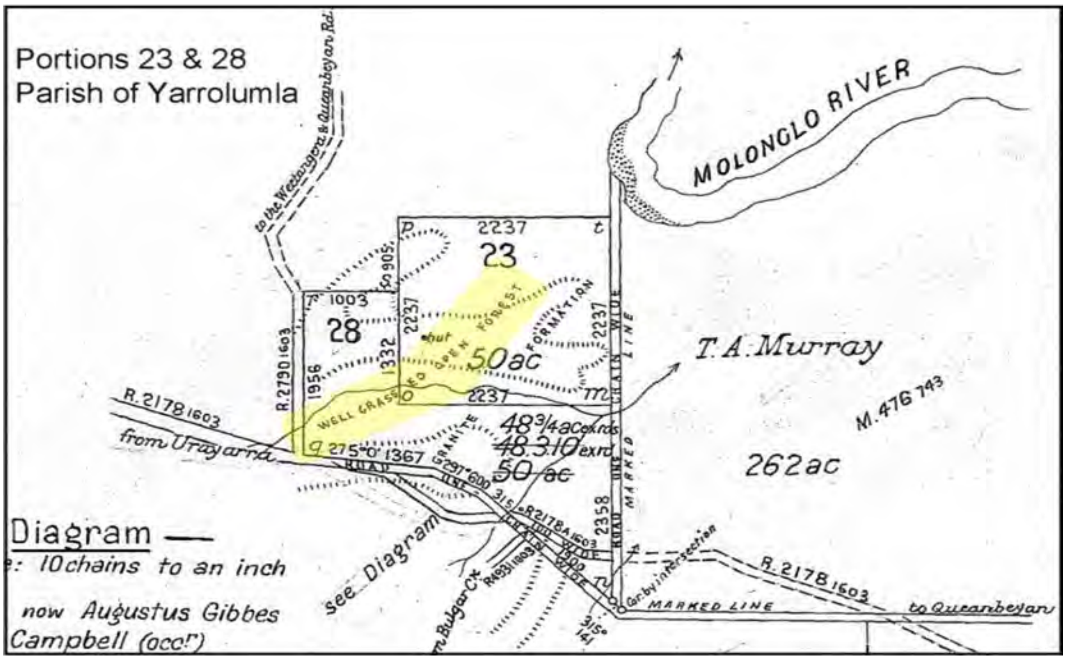

Figure 4: Survey plan for Portions 23 & 28 in the Parish of Yarrolumla (ACT Government Survey Office).

It should be noted that Portions 10, 94 and 97 in the Parish of Weetangera were surveyed in the 1830s and do not include any such notations (Table 2).

Table 2: The portion plan annotations for the original NSW portions in the Parishes of Yarrolumla and Weetangera (Ingwersen, 2007).

Portion No.	Current Block	Survey Year	Relevant Improvements	Annotations	Remarks
Parish of	**Yarrolumla**				
104	1171,1187	1880		Open Forest	
105	1171,1187	1880		Open Forest	
107	1171,1187	1880	Clearing $30	Good Open Forest	See Fig. 5 above
106	West 1187	1880		Open Forest	
11	1171	1855		Dry brow	Dry topographic situation
34	1171	1874	Clearing, hut and yard $45	Nil	
23	1171	1871		Well grassed open forest	See Fig. 4 above
28	1171	1871		Well grassed open forest Granite formation	See Fig. 4 above
7	428 181 South	1837		Description 18/4/1837	
Parish of	**Weetangera**				
10,94,97	181 North	1835		Nil	

3.2.2 Estimating the Maximum Tree Density using Portion Plans

Ingwersen, to support his estimate of 17 to 30 trees per hectare based on Cold Air Drainage limitations and a vegetation type of Woodland, used an accepted formula to estimate the maximum tree density for woodland areas. The formula was first used to calculate tree densities in North America and later use by Lunt (1997) to estimate tree densities in the Gippsland of Victoria in the 1970s:

$$TD = 10000/4d^2 \text{ trees per hectare} \quad (1)$$

where TD is the tree density and d is the average reference distance from the reference tree in metres.

Ingwersen quotes Lunt (1997) to state "these are magnitude estimates rather than precise figures". Given a number of portion corners did not show a tree as a reference mark, this is consistent with Lunt's methodology. Lunt's estimates may vary between 2 and 59 trees per hectare across a range of environmental conditions.

From Table 3, the maximum tree density computed using the relevant portion plans is 36 trees per hectare. This differs slightly from the figure of 34 trees per hectare given by Ingwersen in the table at Appendix 2 of his report. Ingwersen, in his calculations, did not include one tree and a reference bearing was used in place of a reference distance, thereby distorting the result slightly. Nonetheless, the result shown here strongly supports Ingversen's estimate of 17 to 30 trees per hectare.

Table 3: Tree density calculation from portion plans in the vicinity of the lessees blocks as revised (Appendix 2 from Ingwersen, 2007).

Parish of Yarrolumla								
Portion (Por)	Corner	Defined at corner	Reference Distance Links	Tree Type	Included	Why Excluded (same as)	Reference Distance Metres	Distance Used Metres
7	A	At corner	0	Oak Sapling			0.0	
	B		30	Gum	Yes		6.0	6.0
	C	At corner	0	Stringy bark	Yes		0.0	
	D	At corner	0	Large oak			0.0	
11	A	At corner		Peg			0.0	
	B	At corner	0	Gum	Yes		0.0	0.0
	C		26	Gum	Yes		5.2	5.2
	D			Large Oak			0.0	
23 & 28	M		48	Gum	Yes		9.7	9.7
	N			No tree	Yes			
	O	At corner		Apple	Yes		0.0	0.0
	P		41	Gum	Yes		8.2	8.2
	Q		16	Gum	Yes		3.2	3.2
	R		41	Gum	Yes		8.2	8.2
	S		31	Gum	Yes		6.2	6.2
	T		65	Apple	Yes		13.1	13.1
34	A		31	Gum	No	Por 28 S	6.2	6.2
	B		16	Gum	Yes		3.2	3.2
	C		30	Gum	Yes		6.0	6.0
	D		31	Apple	Yes		6.2	6.2
104 & 105	A		65	Apple		Por 23 T	13.1	
	B		41	Gum		Por 23 P	8.2	
	C		31	Apple		Por 34 D	6.2	
	D		56	Apple	Yes		11.3	11.3
	E		35.5	Stringy bark	Yes		7.1	7.1
	F		52	Dead	Yes		10.5	10.5
	G		131.5	Gum	Yes		26.5	26.5
	H		57	Gum	Yes		11.5	11.5
	J		48.5	Stringy bark	Yes		9.8	9.8
106	A		57	Apple	Yes		11.5	11.5
	B		10	Stringy bark	Yes		2.0	2.0
	C		84.5	Gum	Yes		17.0	17.0
	D		55.5	Dead	Yes		11.2	11.2
	E		15.5	Gum	Yes		3.1	3.1
	F		33.5	Gum	Yes		6.7	6.7
107	A		52	Dead	No	Por 104 F	10.5	
	B		47	Oak Sapling	No		9.5	
	C		55.5	Dead	No	Por 106 D	11.2	
	D		84.5	Gum	No	Por 106 C	17.0	
	E		10	Stringy bark	No	Por 106 B	2.0	
	F		75	Gum	Yes		15.1	15.1
	G		57	Gum	No	Por 105 H	11.5	
	H		131.5	Gum	No	Por 105 G	26.5	
Parish of Weetangera								
97				No marked trees				
10				No marked trees				
94				No marked trees				
					Average Distance			8.3
				Estimated Maximum Tree Density				36.062404

Lunt (1997) notes three qualifications to this methodology:

1. It assumes there was no large-scale clearing of trees between the time of settlement and the survey for the relevant portion. Lunt notes there is a 20-year gap between settlement in the 1840s and the survey in his area of interest being the Gippsland. However, the Molonglo Valley was settled some 40 to 50 years prior to the relevant portion surveys.

2. It assumes trees are randomly dispersed rather than clumped. Some clumping may occur due to competition between tree species and local climate variations.
3. It assumes surveyors always go for the closest suitable tree. Some evidence indicates surveyors favoured certain sized and species of tree.

In addition, survey reference trees need to be of a size adequate to cut a shield into it. This may distort the calculations somewhat.

3.2.3 Tree Cover from the 1910s to 2005 and Usage of Early Mapping & Aerial Imagery

In order to determine the extent of tree cover from the time of the Commonwealth Territory's establishment to when relevant leases commenced and later expired, Ingwersen relied upon mapping and aerial photography of the Molonglo Valley supplied by the ACT Government Survey Office.

In 1914, at the establishment of the Commonwealth Territory, the Federal Capital Feature Map of holdings was produced. The map included many annotations about the state of the vegetation and showed the observed tree line (Figure 5).

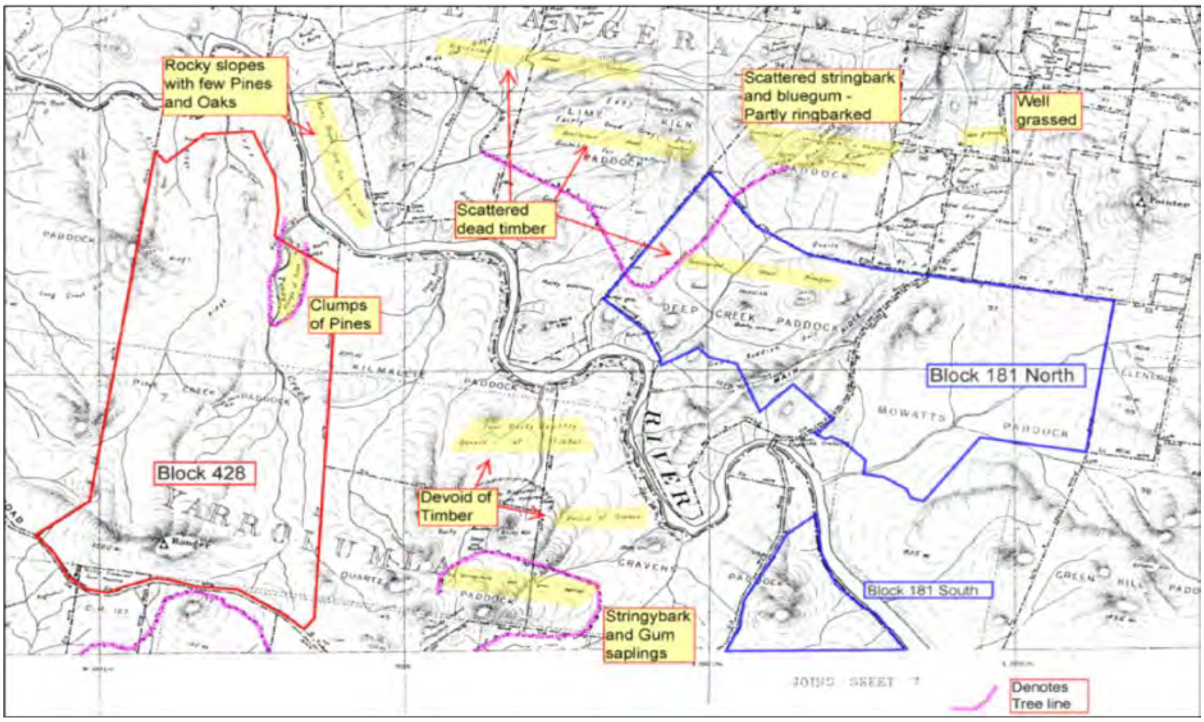

Figure 5: Part of the Federal Capital Feature Map, Sheet 3 (1914) (ACT Government Survey Office). The vegetation annotations have been reproduced and enlarged in text boxes for clarity.

Ingwersen relied on the Feature Map annotations to contend that the subject blocks had undergone extensive clearing prior to 1914. The annotations described the vegetation as scattered, dead, ringbarked or in small clumps. Interestingly, the tree line across Block 181 North is in much the same position as shown in the 1959 and 2004 aerial photography. The Feature Map, Sheet 7 to the south did not contain the same level of annotations.

Most original leases commenced in the 1920s and 1930s, which is less than 20 years after establishment of the Commonwealth Territory. Ingwersen argued that further clearing prior to the commencement of the first leases was unlikely in part due to the efforts of the Federal Capital Commission and the Timber Protection Ordinance (Commonwealth of Australia Gazette, 1919).

Table 4 shows the lease commencement dates, which were important with respect to Clause 2(c) of each lease, referred to previously. The most recent leases commenced in 1956 and expired at the end of 2005.

Table 4: Lease commencement dates for all three properties.

Lease	Leased Commenced	Lease term (Years)	Block/s	District	Deposited Plan Number	Notes
Coonan	1 Dec 1938	0.25	47	Belconnen		Lease
	1 Jul 1933	25	39	Belconnen	136	Lease
			43	Stromlo		
	1 Jan 1956	**50**	39 & 47	Belconnen	673	Lease
			43	Stromlo		
Tanner	1 Feb 1926	11	10	Stromlo	10	Lease
	1 Jul 1933	25	10	Stromlo		Lease
	1 Jan 1956	**50**	10	Stromlo		Lease
	15 Jul 1992	N/A	428	Stromlo	8114	Balance Crown Lease
Tully	1 Mar 1927	5	15	Woden		Lease
	1 Jul 1933	5				Lease
	6 Sep 1938	5				Lease
	1 Jan 1956	**50**	15	Woden	712	Lease
			65	Belconnen		
	15 Nov 1999	N/A	1171	Weston	9456	Balance Crown Lease
			1187	Creek		

Aerial imagery was made over the subject blocks in 1952, 1953, 1955, 1959 and 2004, which covers the period of the most recent leases. The 1952, 1953 and 1955 aerial imagery, though only partially covering the subject blocks, shows the same tree cover as the 1959 imagery. Furthermore, the 1959 and 2004 aerial imagery was consistent in showing virtually no loss of tree cover over nearly 50 years. In fact, Block 428 appears to show some regrowth. All the aerial imagery shows a similar tree line, which approximates the limit of CAD effects determined by Ingwersen (Figures 6 & 7).

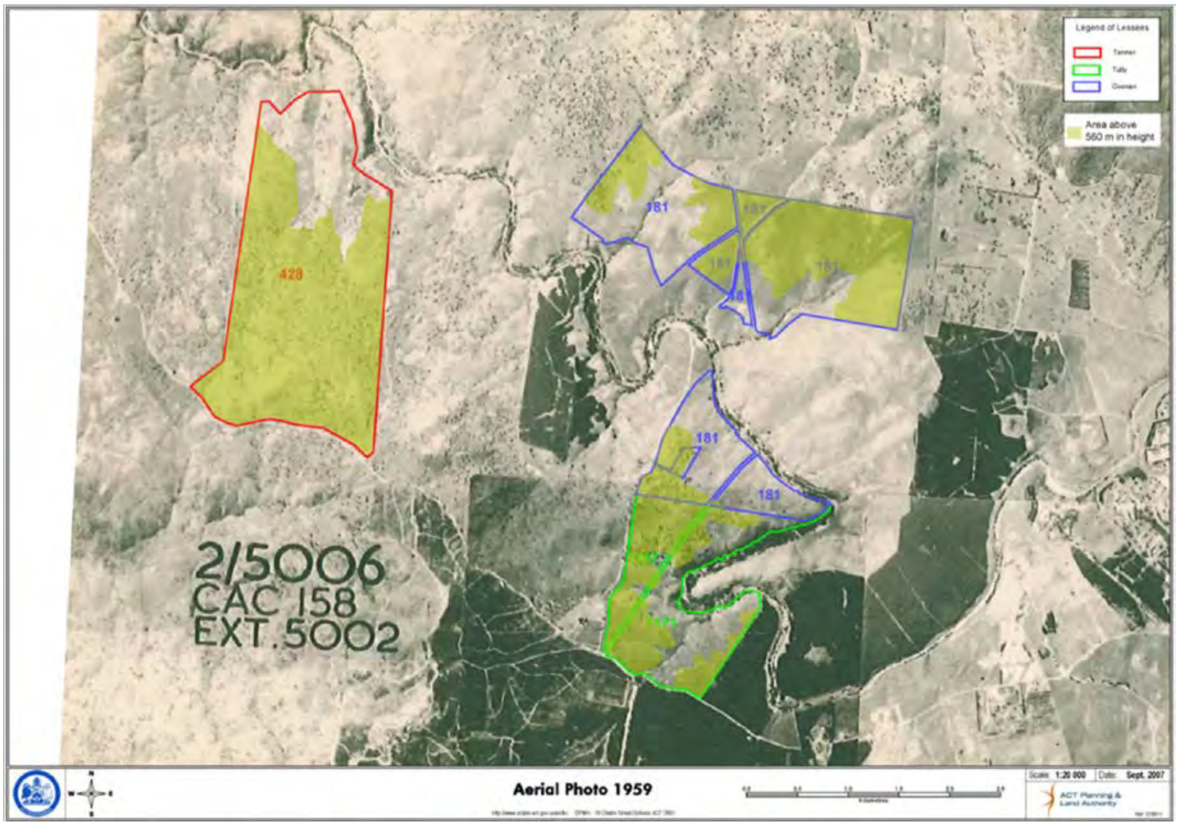

Figure 6: The 1959 aerial image with the subject blocks (ACT Planning and Land Authority, 2007).

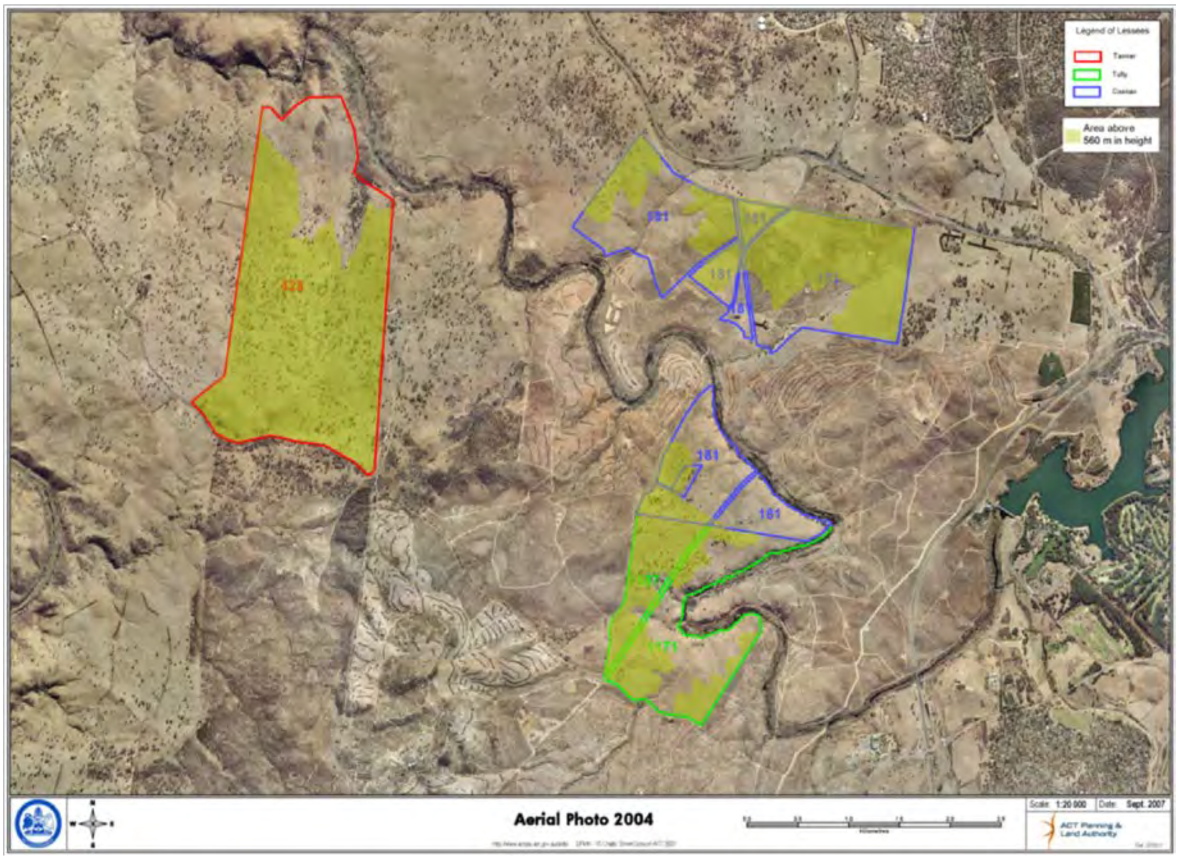

Figure 7: The 2004 aerial image with the subject blocks (ACT Planning and Land Authority, 2007).

4 CONCLUDING REMARKS

The Cumberland Report argued for tree densities of 200 to 300 trees per hectare. While claiming to be based on historical evidence and the adopted appropriate tree classifications, the figures did not appear to take account of the best scientific or historical evidence available.

In his response, Ingwersen (2007) addressed the temporal nature of the tree cover question by drawing on scientific evidence, scientific accounts written as early as 1910, NSW portion plans, mapping terrestrial imagery and aerial imagery.

Ingwersen's report demonstrated:

1. Cold Air Drainage had a long-term effect in suppressing the establishment of trees below the 540 to 560 m elevation limit. Only a small number of cold-tolerant trees had established themselves below these limits.
2. The original vegetation cover in areas above the 560 m elevation limit would, consequently, be best described as Woodland or Open Forest with a tree density of 17 to 30 trees per hectare based on the appropriate vegetation classification from the Atlas of Australian Resources, Vegetation Map (AUSLIG, 1974).
3. The early European settlement period was characterised by tree clearing as evidenced by portion plan notations from the 1880s, contemporary accounts and painting, and the Federal Capital Feature Plan of 1914.
4. Maximum tree densities on the subject properties were about 34 to 36 trees per hectare based on the method which used the reference distances shown on portion plans. The maximum density is consistent with the 17 to 30 trees per hectare estimate made.
5. Further clearing was unlikely through the period of early leases from the 1920s to the 1950s, due to a strong mitigating influence by the Federal Capital Commission and the Timber Protection Ordinance of 1919 to 1940.
6. The 1950s aerial imagery was consistent with the Federal Capital Feature Plan of 1914.
7. Little or no tree clearing occurred during the period of the most recent leases from 1956 to 2005 as evidenced by the aerial imagery produced in 1952, 1953, 1955, 1959 and 2004.

Ingwersen effectively argued the tree densities, on the leased properties, were only about 15% of what had been claimed by in the Cumberland Report and little tree clearing had been effected pursuant to Clause 2(c) of the leases, by either a lessee or a previous lessee from the 1920s to 2005. As a result, the arbitration decision was in favour of the ACT Government.

ACKNOWLEDGEMENTS

The input provided by Ron Jarman, ACT Deputy Surveyor-General, in regards to this paper is gratefully acknowledged.

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A Cadastre Set in Stone

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ABSTRACT

At North Ryde, in 1881-82, Mr Surveyor Charles Robert Scrivener undertook a survey within the Field of Mars Common to create 125 Portions and 51 Suburban Allotments (an area equivalent to one twelfth of the current area of Ryde!). 25 of the streets created by this survey still exist. This plan of survey was catalogued as 386.2030 in the Lands Department plan system. Like many other survey plans in this system, with the constant handling, constant notations and updating, together with material ageing, Crown Plan 386.2030 deteriorated, faded, lost information and became almost indecipherable. The current image of 386.2030 shows multiple areas of the plan have been lost and shows past attempts at stabilising the plan onto a solid backing with subsequent errors in replacement of loose pieces. The current image even shows a large black stain from an historic ink spill. This paper documents the re-creation of a disintegrating 1881 survey plan and the re-instatement of the original street pattern with reference to found original survey marks.

KEYWORDS: *Original marks, alignment, preservation, re-instatement*.

1 INTRODUCTION

At North Ryde, in 1881-1882, a Lands Department Staff Surveyor, Mr Charles Robert Scrivener, undertook a survey within the Field of Mars Common to create 125 Portions of generally 4, 5 and 6 acres each and 51 Suburban Allotments – an area equivalent to one twelfth of the current Ryde Local Government Area!

Granted by Governor King in 1804, the Field of Mars Common was an area of Crown Land that extended along the southern side of the Lane Cove River from Hunters Hill to Pennant Hills (Figure 1). In the tradition of the English Common, it was for the supplementary use of the local residents. The Common also effectively preserved much of the native bushland along the Lane Cove River and covered an area of 2,044 ha. By 1875, the Government proposed to resume the Common, with parts being sold off in order to raise moneys for the funding of infrastructure projects such as the building of the Iron Cove Bridge and Gladesville Bridge which would provide a more direct road access from Sydney Cove to the market gardens and farms of Marsfield and Ryde.

The survey plan prepared by Scrivener was catalogued in the Lands Department plan system as 386.2030 (Figure 2). The portion numbers commenced at 201 (only 150 portions had previously been created in the parish) and 25 of the streets created by this survey still exist. The purpose of this paper is to outline the re-construction of Crown Plan 386.2030 and the re-establishment of the 25 streets using original marks, as placed by Scrivener in 1881-82, and as found by City of Ryde survey team in 2016, i.e. 135 years later!

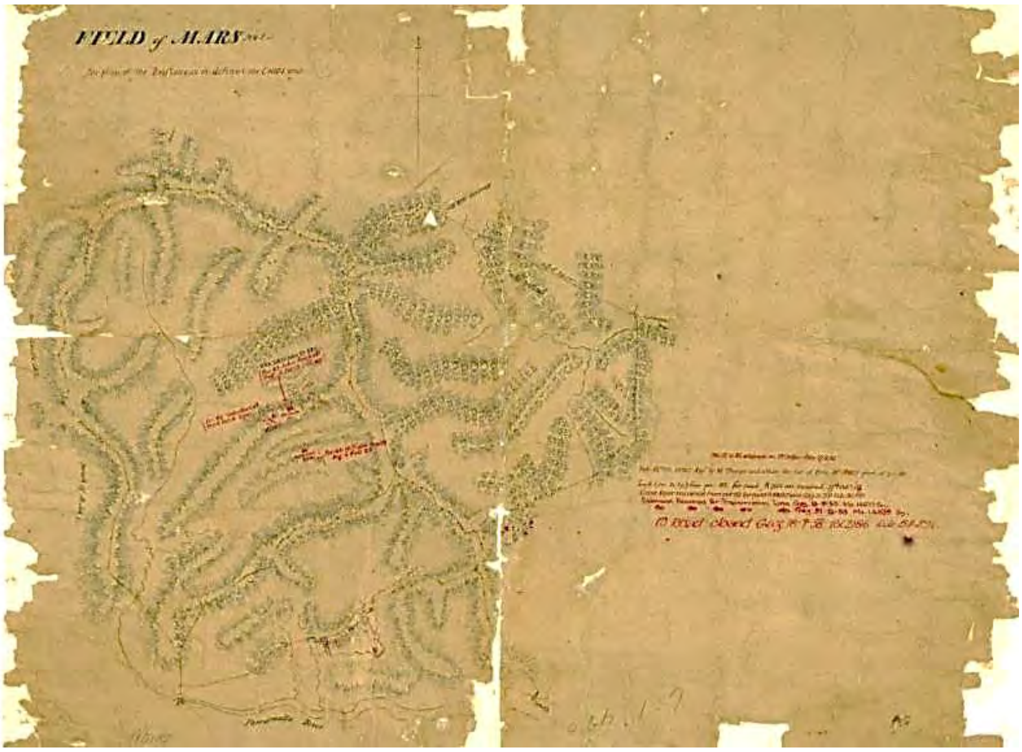


Figure 1: Crown Plan 1.480 from 1804 showing the Field of Mars.

Figure 2: Crown Plan 386.2030. Date of survey from September 1881 to May 1882.

2 RE-CONSTRUCTING CROWN PLAN 386.2030

Like many other very old survey plans in the Lands Department system, with the constant handling, constant notating and updating and material ageing, Crown Plan 386.2030 has deteriorated, faded, lost information and became almost indecipherable. These were the days before microfilming and electronic data capture! The current image of Crown Plan 386.2030 clearly shows the poor state of the plan, where multiple areas of the plan have been lost, and shows past attempts at stabilising the plan onto a solid backing; with subsequent errors in replacement of the loose pieces (Figures 3 & 4).

The current image even shows a large black stain from an historic ink spill (Figure 2) and up to seven rows missing off the bottom of six columns in the “Reference to Traverse” table (Figure 5).

One historical snippet which appears on the plan concerns the village of Marsfield. The name Marsfield was obviously concocted from Field of Mars, and a text on the plan (Figure 6) shows “Village of Marsfield” located in the place of modern-day Boronia Park shops. Marsfield is today a suburb 5 km away to the north-west.

Detail of Portion 273 (Figure 7) shows two pieces of the original plan, which have been attached to backing board in incorrect placement. Once these two pieces have been re-arranged (Figure 8), visible text is suddenly much clearer and makes sense. “John Kendall Duguid”, Portion Number followed by area and “one chain wide road”.

A notation on Crown Plan 386.2030 refers to another Crown Plan 36.2063, being “Plan of Proposed Reserves in the Field of Mars Common” (Figure 9).

This “Plan of Proposed Reserves” was carried out in 1881-1882 at the same time as Crown Plan 386.2030 and by the same surveyor. As this survey is a plan of reserves, reference to and use of the original plan has been minimal over the years, so the current quality of the original plan as well as the image from the original plan is very good. The “Reference to Traverse” table on Crown Plan 36.2063 (Figure 10) displays almost exactly the same traverse information shown on Crown Plan 386.2030, which is now lost or indecipherable (see Figure 5).

Consequently, a large part of the control for Crown Plan 386.2030 can be re-established and those missing 7 lines from the table in Figure 5 can be re-instated. However, even with the river and creek traverses known, there is little on the “Plan of Reserves” to help with the individual dimensions for individual land parcels.

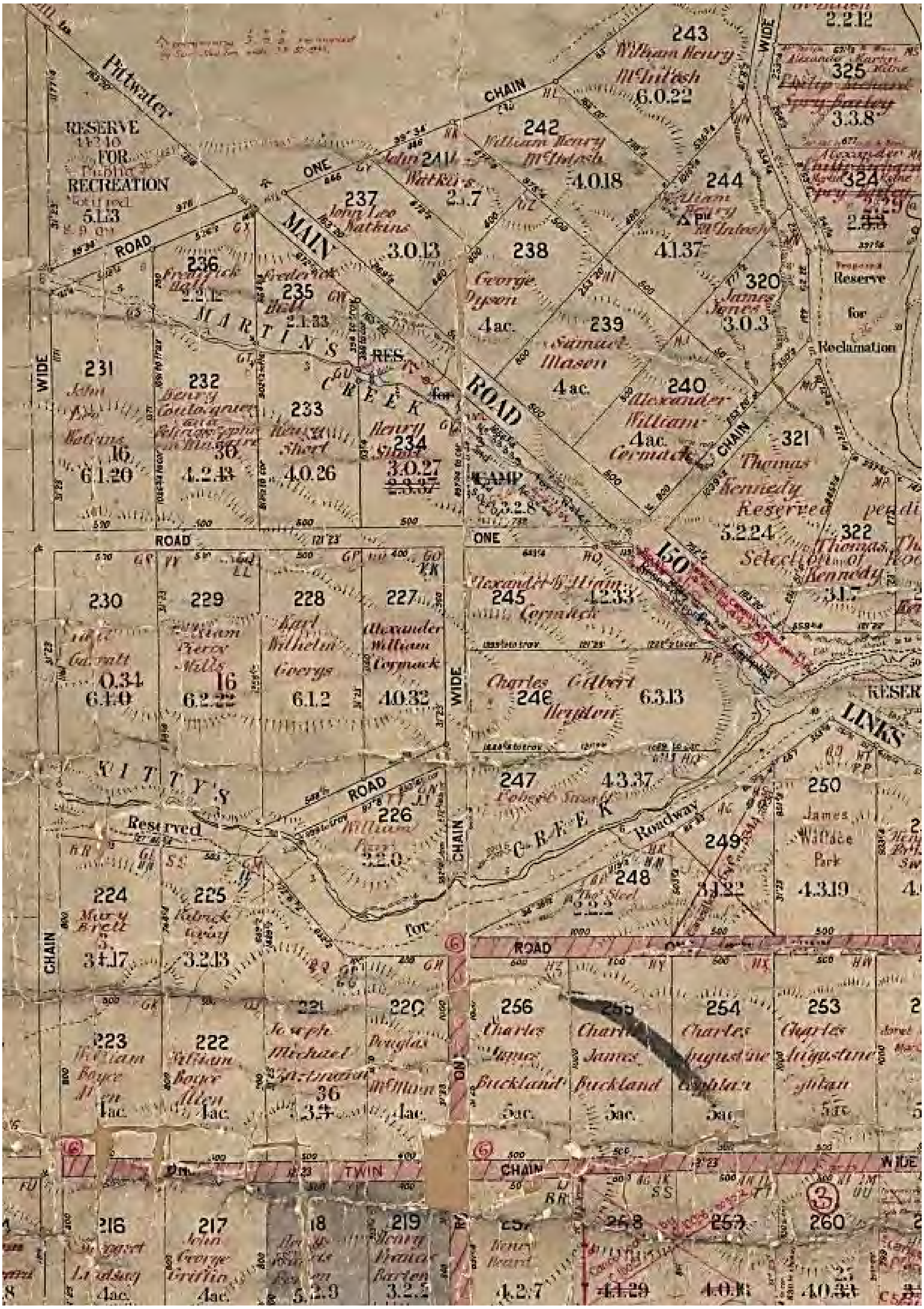


Figure 3: Part of Crown Plan 386.2030, at half scale, covering part of area of investigation.

Figure 4: Part of Crown Plan 386.2030, at half scale, covering part of area of investigation.

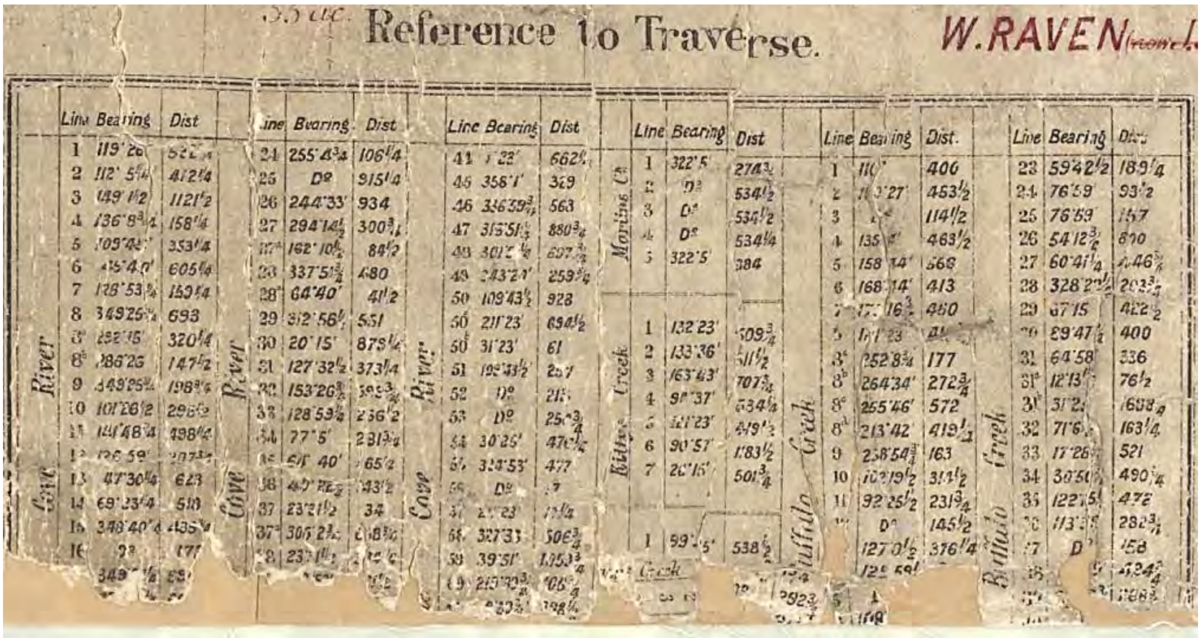


Figure 5: Up to 7 lines missing from the bottom of the traverse table.

Figure 6: “Village of Marsfield” sited where modern-day Boronia Park shops are situated.

Figure 7: Detail showing part of plan fragments as attached by the Lands Department.

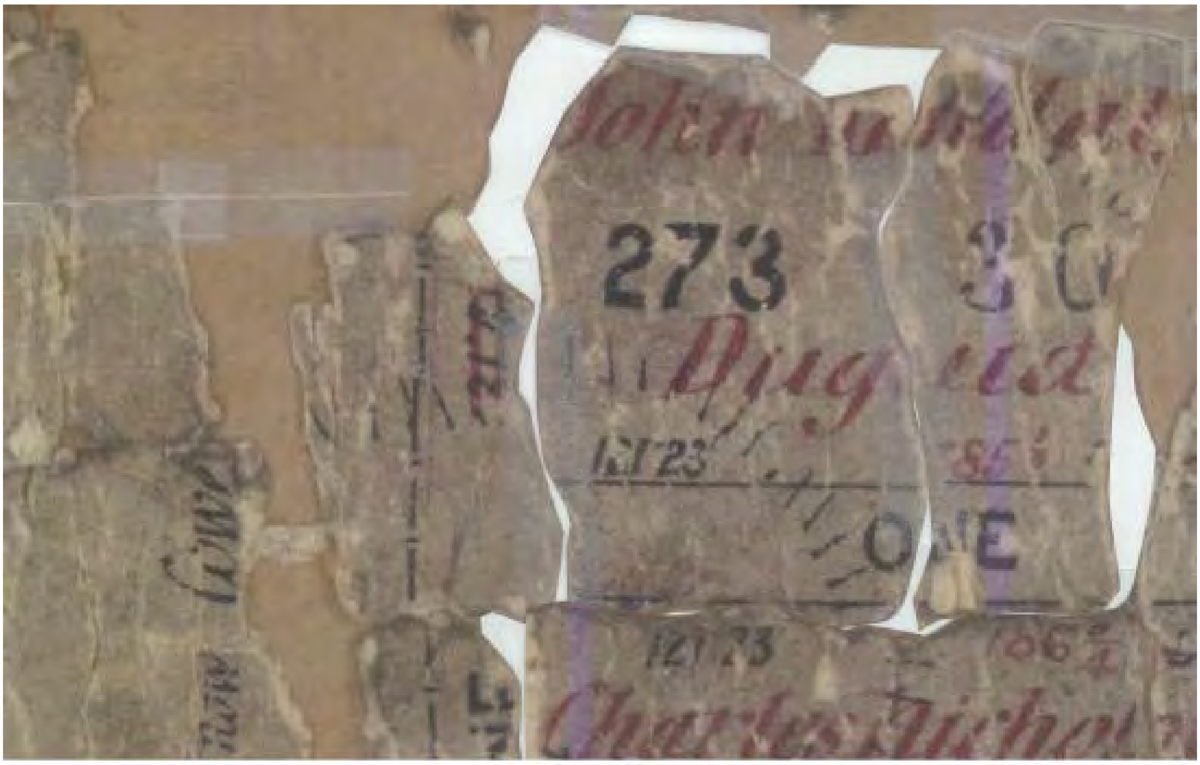


Figure 8: Detail showing the fragments restored to their correct position.

Figure 9: Crown Plan 36.2063 (1881-82) of "Proposed Reserves in the Field of Mars Common".

Line	Bearing	Distance	Line	Bearing	Distance	Line	Bearing	Distance
1	49° 40'	605¼	31	153° 26¾'	599¾	56	39° 51'	953
2	126° 53¼'	159¼	32	128° 59¼'	256½	57	322° 5'	274¾
3	349° 25¾'	52	33	77° 5'	281¾	58	Do.	534½
4	Do.	641	34	64° 40'	365½	[illegible]	99° 35'	538½
5	Do.	198¾	35	40° 22¾'	243½	60	Do.	430¾
6	101° 26½'	295½	36	23° 31½'	34	61	Do.	107¾
7	141° 48¼'	498¼	36a	306° 2¾'	658¾	62	211° 23'	37¼
8	126° 59'	207¾	37	23° 31½'	384¼	63	137° 24½'	520
9	47° 30¼'	623	37a	130° 53½'	661½	64	107° 55'	514
10	69° 23¼'	518	37b	7° 43¾'	575	65	117° 51'	501
11	348° 40¾'	318½	37c	320° 28¼'	721½	66	Do.	Do.
12	Do.	111¼	38	30° 15½'	300	67	Do.	Do.
13	Do.	172	39	16° 52¾'	215¼	68	31° 23'	129
14	349° 27¼'	694¼	40	337° 17'	247¼	69	170° 16¾'	460
15	326° 54¼'	603¾	41	272° 25¾'	766	70	121° 23'	410½
16	343° 28½'	425	42	292° 59¼'	527¼	70a	252° 8¾'	177
17	359° 27¾'	703¾	43	1° 23'	662¼	71	258° 54¾'	163
18	284° 52¼'	687	44	358° 1'	329	72	102° 19½'	314½
19	305° 26¾'	479¾	45	336° 59¾'	568	73	92° 25½'	231¾
20	313° 56½'	653½	46	316° 51¾'	880¾	74	Do.	145½
21	255° 4¾'	106¼	47	301° 51¼'	697¾	75	127° 0½'	376¼
22	Do.	122¼	48	343° 20'	253¾	76	125° 59½'	1¾
23	Do.	793	49	109° 43½'	928	77	54° 12¾'	800
24	244° 33'	934	49a	211° 23'	694½	78	60° 41¼'	446¾
25	294° 14½'	300¾	50	109° 43½'	470	79	328° 22¼'	203¾
26	337° 51¾'	480	51	Do.	250¾	80	67° 15'	422½
27	312° 56½'	551	52	30° 26'	476¼	81	89° 47½'	400
28	20° 15'	879¼	53	324° 53'	494	82	64° 58'	336
29	127° 32½'	76	54	211° 23'	17¼	83	71° 6½'	163¼
30	Do.	302¼	55	327° 33'	506¾	84	17° 28½'	521

Figure 10: Crown Plan 36.2063, "Reference to Traverse Table".

Further notations on Crown Plan 386.2030 refer to three other Crown Plans:

- 15.440 in November 1883, being "Plan of Survey of Field of Mars Common and Grants Adjacent Thereto" (Figure 11).
- 722.2030 in June 1885, being "Amended Subdivision of Part of Field of Mars Common Subdivision", which creates a new road and re-configures Portions 270, 276, 277 and 278.
- R23.2113 in September 1885, being "Tracing Shewing Alignment of Field of Mars Common" (Figure 12).

Each of these plans was done by Charles Robert Scrivener. So, all of the relevant survey work done in this area between 1881 and 1885 was carried out by the one surveyor.

Figure 11: Detail from Crown Plan 15.440 of 1883.

Figure 12: Detail from Alignment Plan R23.2113 of 1885.

Crown Plan 15.440 covers a very large area, from Hunters Hill to Pennant Hills, and shows connection to the trigonometric surveys of the day. Trig mark "O12" is noted (see Figure 11), together with mark "828" which has now been replaced by a State Survey Mark (SS23023).

The network of trig stations followed the high points of the hills as well as along the bank of the Lane Cove River.

Alignment Plan R23.2113 shows the alignment marks placed to define the kerb lines in the roads which were created by Crown Plans 386.2030 and 722.2030. Mr Surveyor Scrivener, in this survey, introduced stone for alignment marking, and Ryde is the first locality in the State to use big stone posts (de Belin, 2014). A mere handful (about 12%) of these big stone posts has survived to this day, but they are sufficiently distributed over the whole of the survey area to also assist in the re-construction of Crown Plan 386.2030.

3 FINDING COMMON GROUND

Let us investigate a section of Crown Plan 386.2030 that is of very poor quality (Figure 13). With the aid of the other survey plans (Figures 14-16), it is now possible to rebuild the boundary information for each parcel of land and determine what survey marks were placed and where to seek them, if they still exist. This disparate information, when viewed together, allows us to re-construct the original cadastral boundaries.

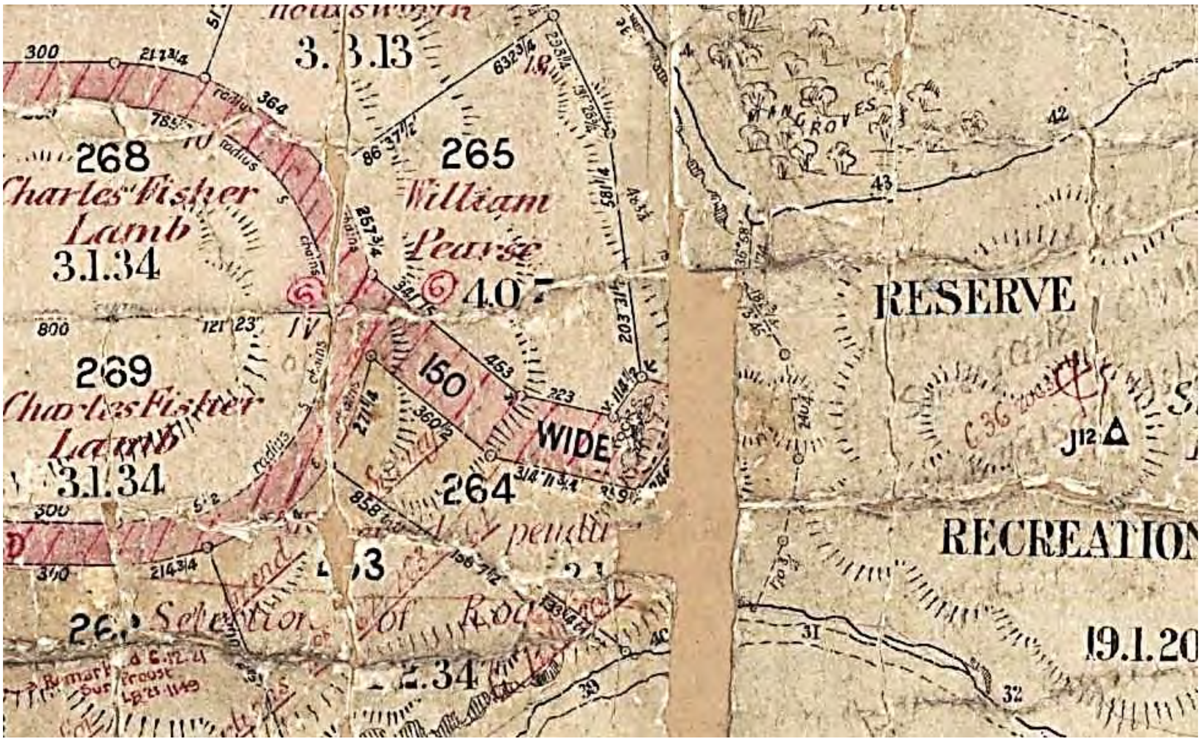


Figure 13: Detail from Crown Plan 386.2030 of 1881-82.

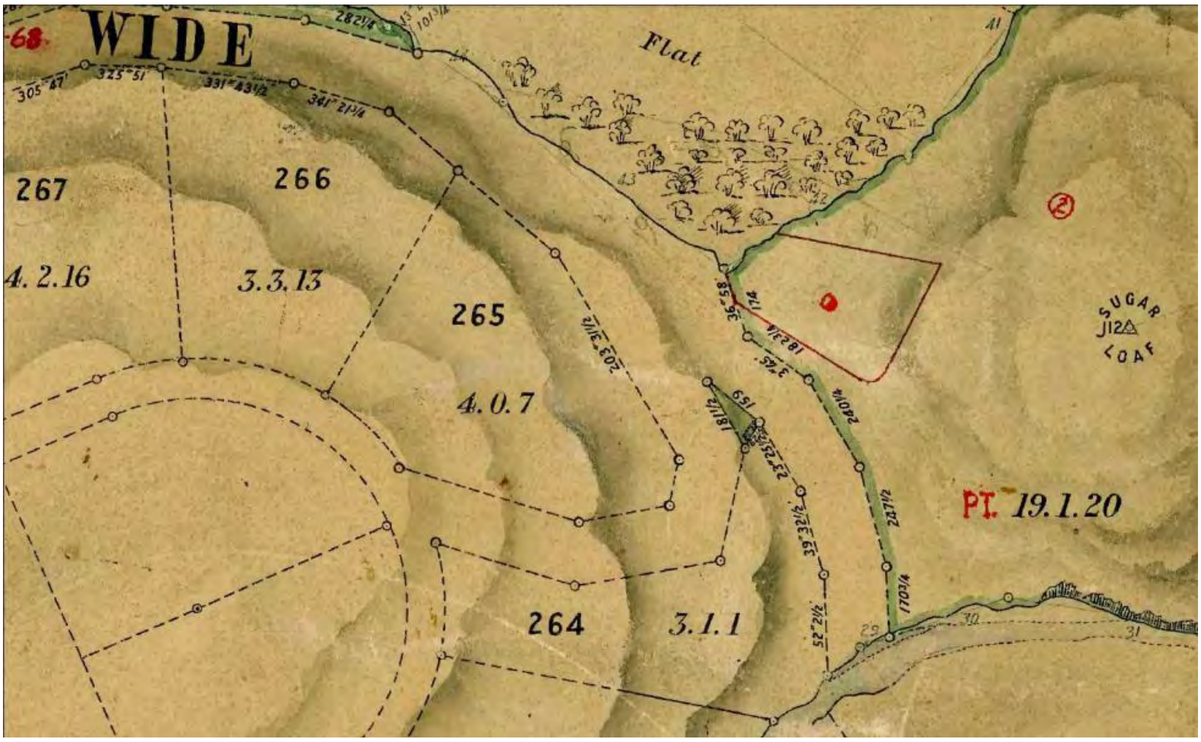


Figure 14: Detail of the same area from Crown Plan 36.2063 of 1881-82.

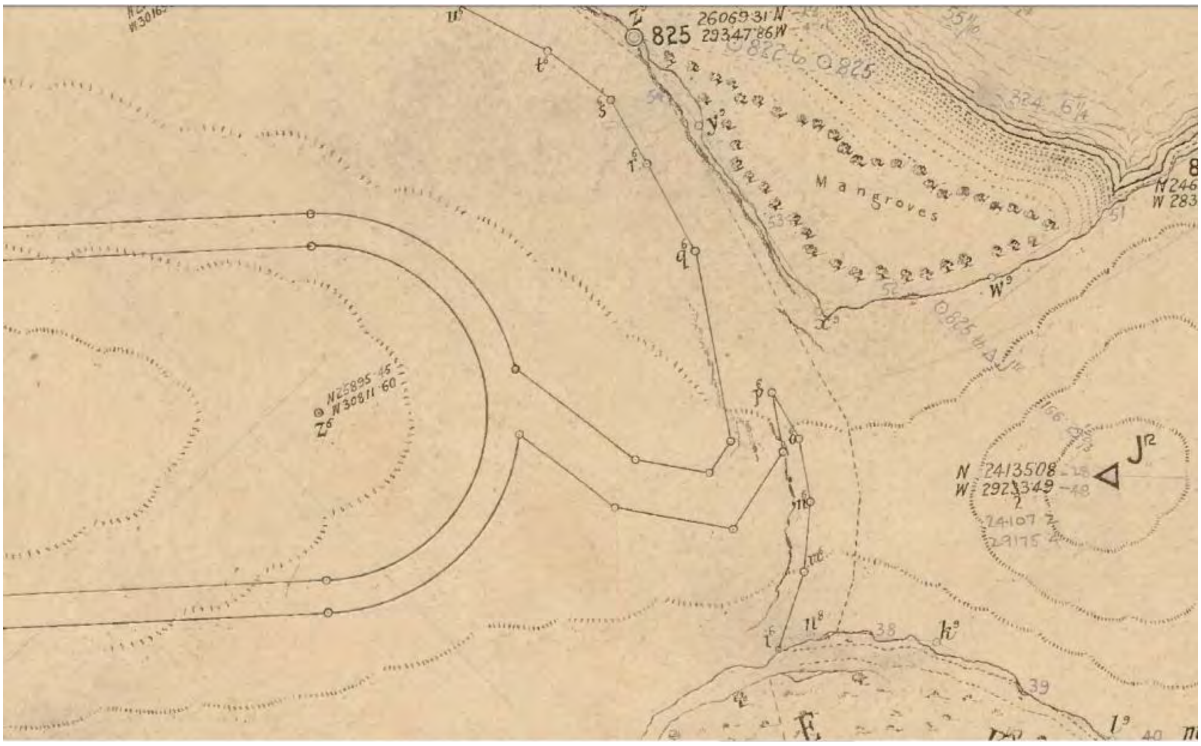
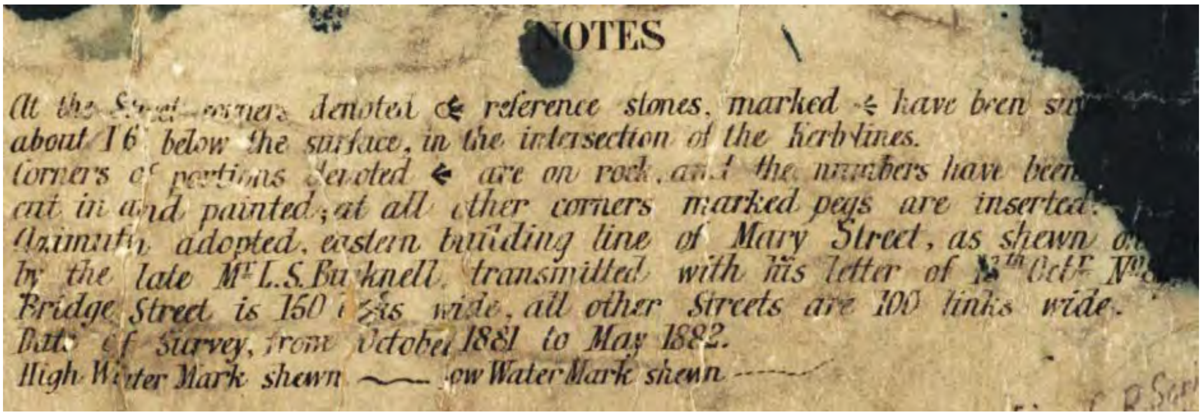


Figure 15: Detail of the same area from Crown Plan 15.440 of 1883.

Figure 16: Detail of the same area from Alignment Plan R23.2113 of 1885.

4 FINDING ORIGINAL MARKS

The survey mark hierarchy rates natural boundary, original mark, monument and then measurement in order of importance in defining land boundaries. So the finding of original marks would be almost irrefutable when it comes to locating an original portion boundary. A perusal of the relevant Crown Plans shows where rock marks were placed and what the nature of these marks was (Figure 17). Part of this note states: “Corners of portions ‘denoted by broad arrow’ are on rock and the numbers have been cut in and painted...”



NOTES

At the Street corners denoted ⇐ reference stones, marked ⇐ have been su
about 16" below the surface, in the intersection of the kerb lines.
Corners of portions denoted ⇐ are on rock, and the numbers have been
cut in and painted; at all other corners marked pegs are inserted.
Azimuth adopted, eastern building line of Mary Street, as shewn o
by the late Mr L.S. Bucknell transmitted with his letter of 13th Octr No
Bridge Street is 150 links wide, all other Streets are 100 links wide.
Date of Survey, from October 1881 to May 1882.
High Water Mark shewn ~~~ Low Water Mark shewn ------

Figure 17: Survey notes from Crown Plan 386.2030 of 1881.

A field search was carried out by the City of Ryde survey team to find any marks which may remain (Figures 18-22).

Figure 18: Detail of Portions 321-323 from Crown Plan 386.2030 of 1881-82.

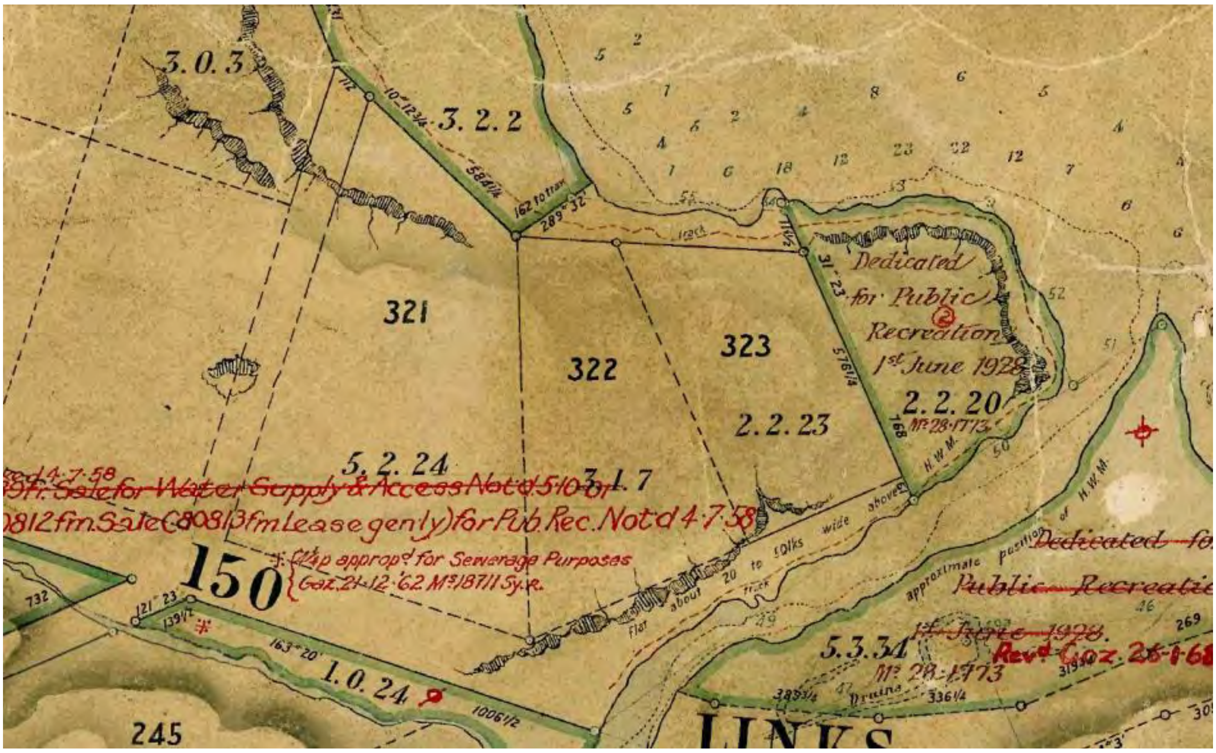


Figure 19: Detail of Portions 321-323 from Crown Plan 36.2063 of 1881-82.

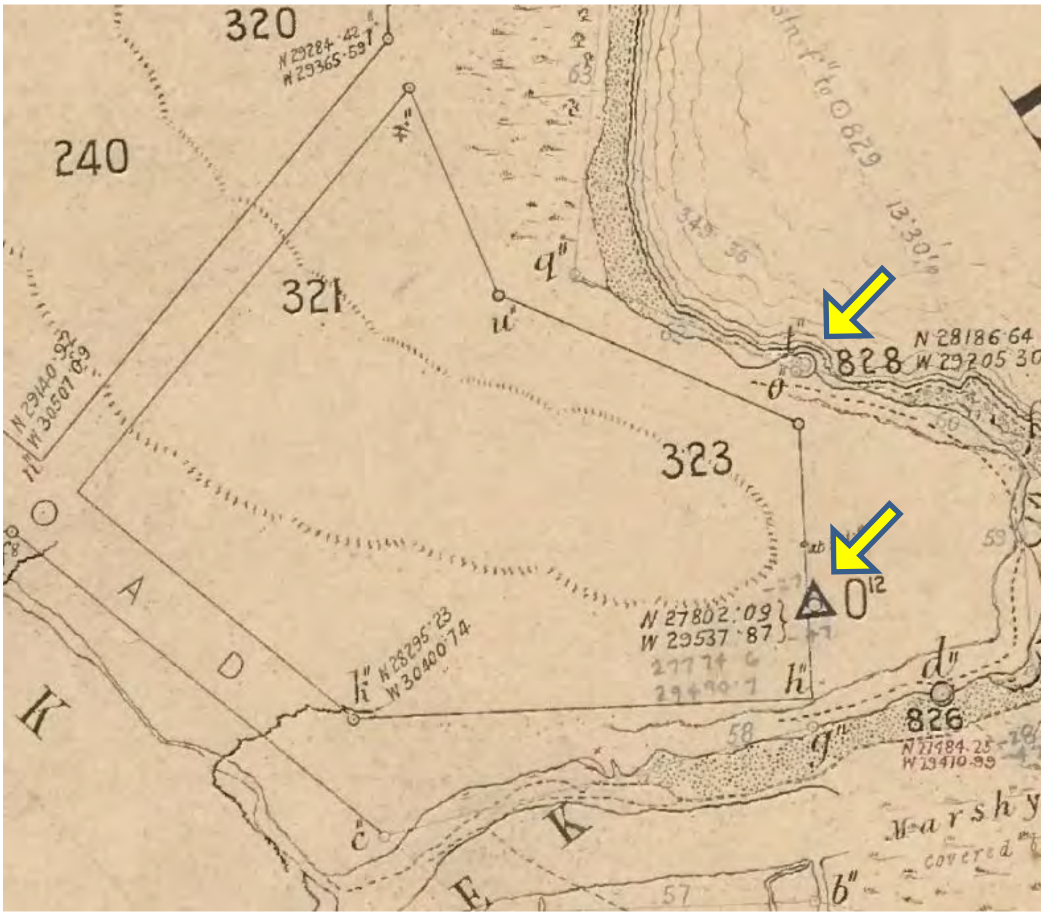


Figure 20: Detail of Portions 321-323 from Crown Plan 15.440 of 1883.

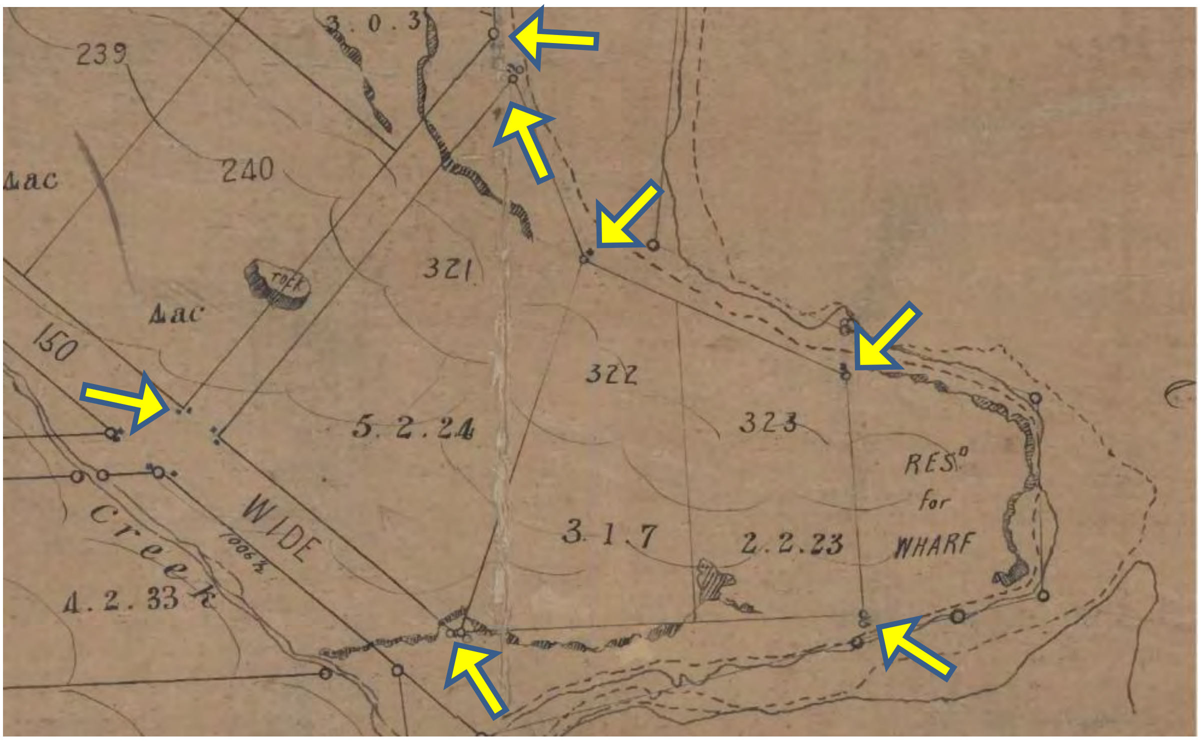
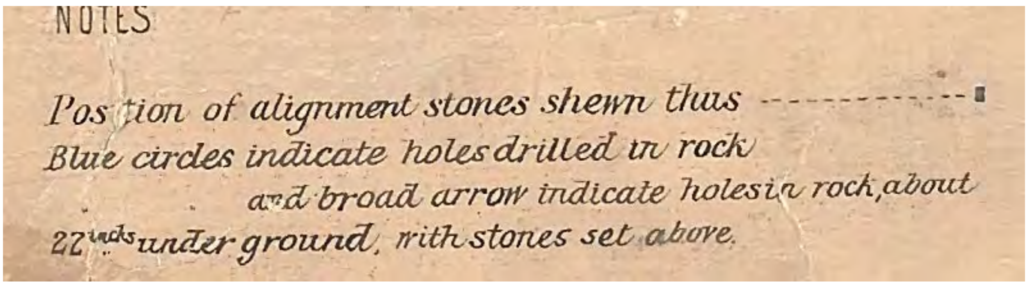


Figure 21: Detail of Portions 321-323 from Alignment Plan R23.2113 of 1885.

Figure 22: Corner rock marks, with chiselled portion numbers, not painted and when painted.

Part of the notes on Alignment Plan R23.2113 indicate alignment stones and holes drilled in rock (Figure 23). Examples of these alignment marks are shown in Figure 24. These alignment marks and corner marks are sited on land which is now under the control of the Lane Cove National Park so have survived the ravages of Council road and drainage works.


NOTES

Position of alignment stones shewn thus

Blue circles indicate holes drilled in rock

and broad arrow indicate holes in rock, about

22 inches under ground, with stones set above.

Figure 23: Survey notes from Alignment Plan R23.2113 of 1885.

Figure 24: Big stone alignment posts and broad arrowed alignment marks in bedrock.

Other rock marks within the survey area include some very old trig stations (Figures 25 & 26). The interested reader can also refer to the Appendix for examples showing the impact of modern residential development on old survey infrastructure.

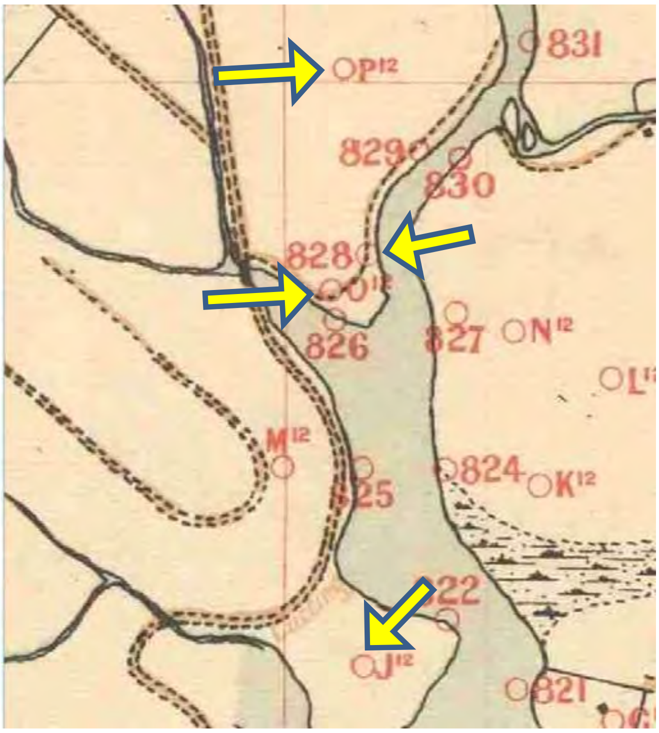


Figure 25: Part of an 1891 publication showing the trig stations in the Sydney area.

Figure 26: Ridgetop trigs 'J12', 'O12' and 'P12'.

5 A CADASTRE SET IN STONE

So far, a total of 50 original marks have been found: 12 corner rock marks, 4 trig station rock marks and 34 big stone alignment posts. It can be clearly seen that these found marks cover a substantial part of Crown Plan 386.2030 (Figure 27).

Figure 27: Sketch showing the location of found marks and the current-day road pattern.

The original street pattern of 25 streets (shown orange in Figure 27) now includes an additional 43 streets (shown green in Figure 27). Half of these newer streets were created prior to 1958, and when first surveyed, were directly connected to original stone alignment posts and marks of the original streets. Since 1959 only 10 of the newer streets have connected to original marks. The last street was created 46 years ago, in 1971.

Undertaking a survey of all the streets over the extent of Crown Plan 386.2030 is a mammoth task and to date a little more than half has been completed. Just how accurate is Crown Plan 386.2030, and what are the comparisons with found marks? Comparisons with the completed survey are shown in Table 1.

Table 1: Comparisons of dimensions from Crown Plan 386.2030 with found marks.

	Per Original	**By Author**
Corner to corner (rock marks)	66º 25’ 45” 1697.24	66º 25’ 45” 1697.470
Corner to corner (rock marks)	81º 49’ 45” 160.935	81º 49’ 40” 160.945
Corner to corner (rock marks)	171º 49’ 45” 171.445	171º 47’ 05” 171.480
Corner to corner (rock marks)	111º 36’ 30” 243.645	111º 37’ 00” 243.685
Street Alignment Cressy Road	211º 23’ 00”	211º 23’ 30” (approx. 1270 m)
Street Alignment Badajoz Road	211º 23’ 00”	211º 23’ 30” (approx. 1270 m)
Street Alignment Quarry Road	121º 23’ 00”	121º 23’ 30” (approx. 380 m)
Street Alignment Forest Road	121º 23’ 00”	121º 23’ 30” (approx. 420 m)

There is a consistent scaling factor of 1.00014 between Crown Plan 386.2030 and the survey carried out by City of Ryde. When applied to the original dimensions, no difference is greater than 10 mm.

6 CONCLUDING REMARKS

Results so far show Crown Plan 386.2030 is accurate to today’s standards and, moreover, is able to be fully replicated and placed within the cadastre, because its position is locked in by original rock marks which can be easily accessed. The fact that so many of the streets are connected to the original marks means that the present cadastral pattern should reflect the original portions and Crown grants. There are now 2,243 lots descendant from the 125 portions surveyed by Crown Plan 386.2030 in 1881, so there are many ongoing benefits in having and maintaining a sound cadastre.

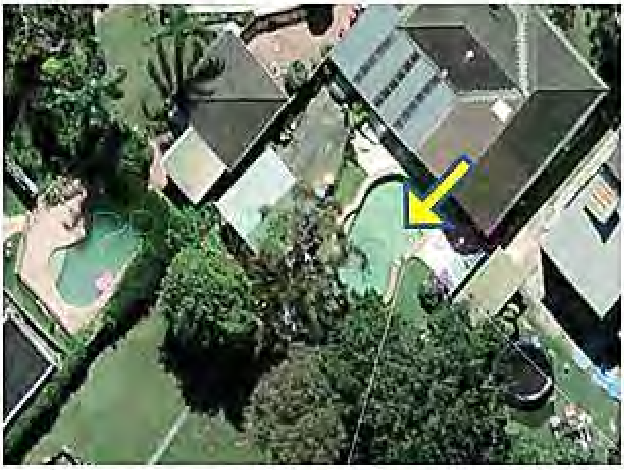
Crown Plan 386.2030 was the first subdivision of the Field of Mars Common. Crown Plan 1156.2030, in 1886, was the second. It abuts on the west and is twice as extensive (creating 227 portions), covering Macquarie Park and Marsfield. The City of Ryde survey team is currently extending its investigation into this adjoining area and the question being asked is how will a new adjustable Map Grid of Australia (MGA) coordinated cadastre fit with the corners of the old cadastre which has been set in stone?

REFERENCES

de Belin F. (2014) Game of stones... The big stone alignment posts of Ryde, *Proceedings of Association of Public Authority Surveyors Conference (APAS2014)*, Pokolbin, Australia, 31 March – 2 April, 115-128.

APPENDIX

These images indicate where several c1881 trig marks are situated in today's world. These sites show a swimming pool, a playing field, a cemetery, a house, a commercial garden centre and a small reserve at West Lindfield. The trig mark in the reserve at West Lindfield still exists and is named "Gordon Trig".



Recent Road Acquisition Surveys in the Unincorporated Area: How do you widen a road that was never surveyed?

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ABSTRACT

Since 2007, the Western Lands Legal Road Network Project has created a defined network of public roads and access easements throughout the Unincorporated Area of NSW. Some 63 plans have been completed where the new road corridors have been created over the existing road formations and road boundaries defined by Map Grid of Australia (MGA) coordinates derived from photogrammetry. Over the last 3 years, Casey Surveying and Design has been engaged by Roads and Maritime Services (RMS) to complete five acquisition surveys to widen, realign and close roads created by that project. This presentation examines each of those five plans, and discusses some of the problems encountered with the Legal Road Network plans and their use of coordinates to define the road boundaries. The presentation also describes the different methods we have used to re-establish MGA at each site and to mark and monument the parcels to be acquired for road, and of the old roads to be closed. Some fascinatingly cadastral finds have been part of these projects too.

KEYWORDS: *Unincorporated area, acquisition, Legal Road Network Project, RMS.*

The First Absolute Gravity Observations in Australia and New Zealand

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ABSTRACT

This paper argues that the first absolute gravity measurements in New Zealand were not carried out in 1882 as in H.W. Robertson & R.A. Garrick (1960), but about 100 years earlier in 1773, just like the first absolute gravity measurement in Australia which was not carried out in 1819 (or even 1937) as in A.S. Murray (1997), but much earlier, in 1788. Huygens' Principle of the reversible compound pendulum dating back to 1673 is used in this argument. It is shown that the gravity measurements of 1773 and later represent absolute gravity after Henry Kater clarified one of the variables. Some adventures of these first gravity field parties are outlined, such as the delay of a survey party by inclement weather and near loss of a survey vehicle, stolen survey equipment, a thief shot at, and field hands allegedly eaten by the locals.

KEYWORDS: *Absolute gravity, reversible pendulum, Shelton clock, New Zealand.*

1 INTRODUCTION

Generally the conventional wisdom is that in the 1800s gravity measurements reached an accuracy of 1/10,000 or 100 milliGals (mGals) and that it was not until the 20th century before there was a gain of an order of magnitude to 1/100,000 or 10 mGals. Henderson (2015) claims accuracies around 3 mGal appeared around 1900. Today we are looking at a few microGals. It was also thought by some that pendulum measurements really needed reference to one other gravity measurement for accuracy and thus still represented relative gravity. The author wants to take a closer look at this, especially the last assertion.

It is thought that the first absolute gravity measurement in Australia was carried out in 1937 but there are some earlier determinations on record. In Murray (1997) there is mention of absolute gravity measurements in Australia in 1819. This fact appears to have been sourced from Dooley and Barlow (1976). In the latter there is indeed reference to French expeditions carrying out pendulum gravity observations. There was gravimetry carried out by Freycinet with a pendulum in Sydney City in 1819 and by Duperrey in Sydney Fort (Fort Denison?) in 1824. But a less well known gravity observation from 1788 predates that, observed with a temperature compensated gridiron pendulum. This work was carried out by a first fleeter: by lieutenant William Dawes, the first fleet astronomer. This observation was mentioned in Morrison and Barko (2009).

About 8 years ago, part of this pendulum gravity observation by William Dawes was brought to the author's attention. This brought about a scoping study in order to evaluate what the pendulum length would have been and whether there was any evidence for this length and

other data. Using the EGM2008 equation for normal gravity, an estimate was made of the pendulum lengths in use by Dawes and others at the time. The search for supporting evidence led to the rediscovery of explanations by William Wales (of the second Cook expedition) about calibrations of pendulums and common errors which can be made using them. A rather serious but recoverable error was actually once made by Wales in setting his pendulum length, so he was speaking from experience. He refers to this in his introduction, in Wales and Bayly (1777).

William Dawes' gravity measurement data was published by the author in Bosloper (2010) at the FIG2010 congress in Sydney. This not only allowed William Dawes' pendulum length to be recovered but opened the door to an understanding of other gravity measurements of 1773 made during Captain Cook's 2nd expedition, for which pendulum calibration information was available. In Chambers (2016) we find a description of Cook's charting of the east coast of New Holland in 1770 during the 1st expedition, but no gravity work was undertaken on this coast according to Howse (1969).

In Maskelyne (1761) the Astronomer Royal Neville Maskelyne, in a letter to Lord Cavendish, described how Dr James Bradley explained to him how to set the pendulum length accurately, in connection with his (Maskelyne's) trip to St Helena for the Transit of Venus. In this letter Maskelyne also refers to the communication in Bradley (1733) by Dr Bradley to the Society, wherein Bradley states that the standard isochronal pendulum length is assumed to be 39.126 "English inches" for London. This value is attributed to measurements by George Graham in 1722. This value was only changed to 39.128 by 1790, almost 60 years later. An isochronal pendulum is a seconds pendulum, marking precise "dead seconds" at a location as determined by stellar transit observations (sidereal time) or noon determinations (for mean solar time). Gravity is basically π^2 times the place's isochronal pendulum length plus a non-linearity correction. This means their gravimetry was carried out with what they knew was physically the right pendulum length and thus the result was absolute gravity for the time. Henry Kater's work later was only an enhancement of the quantification of the pendulum length and can be better related to the French "metre" by a full order of magnitude.

It is now clear that the first absolute gravity measurements in New Zealand were not carried out in 1882 by the U.S. Coast and Geodetic Survey as in Robertson and Garrick (1960), but about 100 years earlier in 1773 by the gravimetry teams of the 2nd Cook expedition. This was 100 years after Jean Richer determined the pendulum length for a seconds pendulum in Jamaica in 1672 and suggested the earth was flattened at the poles.

William Wales' and William Bayly's gravity determinations at Dusky Bay and at Queen Charlotte Sound in 1773 were determinations of absolute gravity. They had timed the swings of the pendulum in Greenwich against an accurate time base, the rotation of the earth, in order to calibrate the pendulum length and set it precisely, right down to the last thousandth of an inch. For example, William Bayly's pendulum calibration showed his pendulum was within three tenthousandth of an inch of the isochronal length for a seconds pendulum at Greenwich when the regulator nut was set with 9 at the index. The only thing that was not yet actually known was how to accurately measure the effective pendulum length of a physical compound pendulum in a practicable way. The fact that the pendulum rod is not without mass and that the mass distribution in the bob is never uniform at the parts-per-million level complicates the matter. The theory which described how to quantify the pendulum length with a reversible pendulum was developed by Christiaan Huygens 100 years earlier and published in 1673, and

a practicable solution came on the scene 40 years after Cook, around 1817 through the work of Henry Kater.

2 THE LENGTH OF THE ISOCHRONAL PENDULUM

In *Horlogium Oscillatorium*, published in Huygens (1673), Huygens developed the "moment of inertia" concept in mathematical terms, and derived an expression for the "reduced length" of a physical (compound) pendulum as a function of the moment of inertia of the oscillating body (Horlogium, Part 4, Proposition VI). It was Leonhard Euler who some 90 years later gave this concept developed by Huygens the name "moment of inertia" in Euler's Theoria Motus Corporum Solidorum sev Rigidorum in 1765.

The "reduced length" is the length between the point of suspension and the point of oscillation of a conceptualised mathematical pendulum with all the mass represented as a point mass at the end of a rigid rod with no mass at all, which oscillates at the same period as the physical pendulum. In *Horlogium*, Huygens showed that any pendulum with the same reduced length will oscillate with the same period when gravity stays unchanged and gave the equation for this.

In his 1673 *Horlogium*, Huygens also developed the parallel axis theorem (Horlogium, Proposition IX) we alternately know as the Huygens-Steiner theorem with which you can recalculate a body's moment of inertia relative to a different axis of rotation or oscillation, parallel to the previous axis. More than a century later the Swiss Jakob Steiner did a more elegant mathematical derivation of this theorem, so it now also carries his name.

In *Horlogium* (Part IV, Proposition XX) we also see the discovery by Huygens that the physical pendulum can be reversed, where the point of suspension is interchanged with the point of oscillation, and the so reversed pendulum will show the same period of oscillation if it is suspended from a parallel axis through the right point. The point of suspension and the point of oscillation will be separated exactly by a distance equal to the reduced pendulum length as defined above. It created the missing step between the conceptual mathematical pendulum and the real physical compound pendulum. This opened a path to be able to determine this conceptual "reduced length" by finding the point of oscillation empirically. The acceleration of gravity can then be determined with the pendulum equation, when the "reduced length" is known.

For a while Huygens' discovery was seen as an academic artefact and other methods of constructing pendulums were tried in order to make the pendulum length more clearly defined and more measurable. Cassini and Borda tried a 12-foot wire pendulum in 1784 with a platinum sphere at the end. Complications arose because the wire flexes laterally during the swings and also stretches and shrinks with temperature and through the changing forces on it.

Then in 1817, Henry Kater found a way to follow Huygens' suggestion. Henry Kater was a member of a committee tasked to refine the London pendulum length. He created a reversible pendulum which contained two opposing knife edges at some separation from each other, which could be rested alternately on a glass plate (Figure 1). Some weights on this pendulum were adjustable so that you could find a position for the weight where the pendulum would swing with precisely the same period in the reversed position. This allowed the distance between the knife edges to be measured to within a few microns and and the pendulum

equation then delivered the absolute value of the acceleration of gravity. In this way, one could now calculate the value for the "reduced length" belonging to any physical pendulum or period.

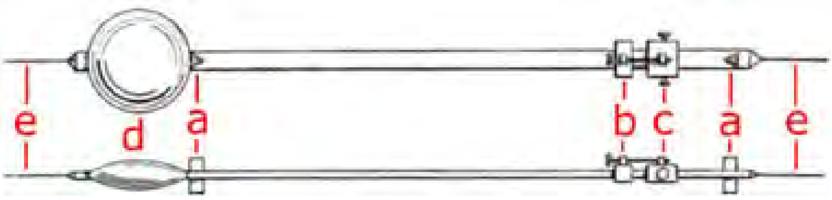

Figure 1: Henry Kater's pendulum (after Watson's physics textbook of 1920). The knife edges are shown as a. The adjustable weights are b & c. The amplitude scale can be read with the pointers shown as e.

It was often thought until now that the earlier period in the late 18th century only produced relative gravity measurements. What one forgets is that the variable (the Greenwich length of the seconds pendulum) which Henry Kater solved in 1817 by constructing a reversible pendulum as described in principle by Huygens, is exactly the length of the pendulums which were used in the previous century. The lengths of the older pendulums were calibrated at Greenwich in the 1700s by the use of astronomy. They were made to swing exactly 86,400 seconds in a 24-hour day by adjusting a regulator nut supporting the pendulum bob – this created an "isochronal" pendulum for that location. Before Kater, the pendulum length itself was *set* at an order of magnitude more accurate than the accuracy with which one could *quantify* the length. This is because the definitions of the point of suspension and the point of oscillation were hard to realise in practice until then. The knife edges now solved this problem.

The 18th century astronomers like Neville Maskelyne at Greenwich were not only able to create and calibrate seconds pendulums empirically, but these bi-metallic gridiron pendulums were also of an intrinsically higher quality than Kater's. This is because of John Harrison's accurate temperature compensation of the body of these pendulums used by Maskelyne. Although Kater later measured the reduced pendulum length to a precision of 2.5 microns, he had to read representative temperatures and calculate the compensation for expansion throughout the duration of the oscillation tests. John Harrison's bi-metallic pendulums of a century earlier were self-compensating for temperature changes in real time.

By inference this means we can use Huygens' conclusions regarding equal periods and say that Maskelyne's pendulums built by John Harrison must have the same reduced pendulum lengths as later determined by Henry Kater if both are made to swing with the same period and have been calibrated against the same time base (the rotation of the earth). Once Captain Cook's pendulum lengths are regarded as known, his astronomers' gravity observations become determinations of absolute gravity at the 10 parts-per-million or 10 mGal level – this is the same order of accuracy with which time could be measured. This means (with 2020 vision) that his determinations in Dusky Bay, New Zealand and Queen Charlotte Sound, New Zealand, and his determinations on Shag Island in Tierra del Fuego, are the very earliest and veritably the *first* absolute gravity determinations in New Zealand and in Chile!

Neville Maskelyne set down some specifications for a clock (or actually a frequency counter) that allowed one to use it as a gravimeter. Before his trip to St Helena in 1761 for a Transit of Venus observation, he made the following list of requirements for an accurate astronomical clock, as mentioned in Higgit (2014): The clock had to have a hand to distinguish seconds and this had to be not only clearly visible but also audible. This already meant it had to include an

approximately 39-inch pendulum so it would swing exactly once every second and cause the "tick". The clock had to experience no loss of time when the once-monthly moment came to wind it up, i.e. it had to maintain 'power'. The pendulum also had to be of a temperature compensated type.

John Harrison had constructed the temperature compensated grid iron pendulum in 1725 as mentioned in Hellman (1931), and had later said he could easily temperature compensate the pendulum clock to not lose more than one second in 100 days if given sufficient funds to enable this. This would have been a phenomenal accuracy of 0.1 parts per million or 100 nanoseconds per second as the accuracy of a time base in the middle of the 18th century, using then current state-of-the-art technology. Even 200 years later the rotation of the earth was still one of the most accurate and reliable time bases. But John Harrison was obviously only allowed sufficient funds to make the astronomical regulator clock pendulum temperature compensated to perform to slightly better than a second per few days or, say, less than 10 parts per million or 10 microseconds per second.

So the gravimeter which these first absolute gravimetry field parties carried was a Shelton astronomical regulator clock. This was basically a very accurate and tall "grandfather clock" with a specially calibrated temperature compensated pendulum which could be given a "reduced length" of g/π^2 metres with g being the gravity at Greenwich, by just adjusting a nut when at Greenwich. This would make it a "seconds pendulum". In order to realise this length, the pendulum was calibrated in Greenwich against the rotation of the earth in such a way that the hands of the clock precisely recorded 86,400 seconds in a mean solar day. This could be accurately fine-tuned with the regulator nut at the bottom of the pendulum.

3 THE FIRST ABSOLUTE GRAVIMETRY IN NEW ZEALAND

In the following story, the mode of transport for the first gravimetry field parties of New Zealand were of course not vehicles but 400-ton 3-masted colliers or coal barks sailing around the world with about a hundred mouths to feed aboard. Maintenance of sails or repairs of a mast or a hull was an often returning occurrence for barks like the Resolution and the Adventure. Gales and fog could separate you, reefs and uncharted rocks were a mortal danger so you mapped them. When some trigonometrical mapping or charting of baselines of a few miles on nearby land were required to help with this, they were not unknown to have been sometimes measured by the "timed gunshot-and-smoke interval" after climbing some of the mountains.

It is amazing that 250 years ago two survey parties could be so confident of their own positioning capabilities to agree that if losing sight of each other they would wait for the other in a bay of a far away island 9,000 km away. Sure enough, they did loose sight of each other after heading north again from the Antarctic South latitude of about 59º, south-east of Cape Town. Somewhere between the Crozet Islands and the Kerguelen Islands two parties were separated by fog, so they stayed in the area for three days as arranged, tacking back and forth and firing a gun into the fog at hourly intervals. The fall back position was to meet at Queen Charlotte Sound in New Zealand, a quarter of a world away. Captain Furneaux, after experiencing frozen sails and brittle rigging and more, headed north with the Adventure after stopping at Van Diemen's Land (i.e. Tasmania) for wood and water and continued on to the Cook Strait of New Zealand. Captain Cook headed for Dusky Bay in the south of the South Island for astronomical work and gravity surveys first, to then also proceed to Queen

Charlotte Sound later.

When the gravimeter had to be deployed, one had to drag a tall grandfather clock, the Shelton astronomical regulator clock, onto a row boat, land it safely after passing through the surf and carry it up to a high and dry spot near the shore. Cook's astronomer William Wales described the Dusky Bay site preparation effort for the gravimetry work in his observations logbook of the Resolution (Cambridge University Library, 2017):

> Thu 25 May 1773. Sailed into Dusky Bay.
> Fri 26. Anchored in 50 fathoms and headed to the place. Found a better Anchoring Place & in the morning moved the ship to it and Anchored there.
> Sat 27. Went in with Capt Cook to look for an observing place & found one; but in the Morning met with Natives, which rendered it improper. Showers.
> Sun 28. Made choice of another. Clearing away the Trees, which grow everywhere here down to the Water's Edge. Heavy Rain.
> Mon 29. Employed as above. Frequent Showers, Wind at North.
> Tue 30. Still Employed cutting down Trees & levelling the Ground. Strong Gales, Northerly & heavy Rain.
> Wed 31. Cutting down Trees and erecting the Observatory. First part Showers: the latter clear & fine weather.
> April 1. Got up the Clock and Quadrant but found they would not do, the Ground was so loose. First part Clear, Latter, showers as usual.
> Apr 2. Cut down two large trees which stood close together, on the stump of one set my Clock, on the other the Quadrant and Erected the Observatory over them.

The Bird quadrant referred to above, as well as delivering latitude and longitude, was for determinations of mean noon by equal altitudes of the sun in order to determine the clock rate of the astronomical regulator clock which was the actual gravimeter.

This site preparation for New Zealand's first absolute gravity measurement had taken the first seven days after their arrival. The preparations for stabilising the regulator clock were once described in Dixon (1769) and Bayley (1769). Generally, a 3-foot deep hole would be dug in which a 14-inch wide board or oak plank, possibly at least 6-foot long and almost 5-inch thick would be inserted, standing up perfectly vertical. This is almost as massive as two railway sleepers shoulder to shoulder and almost one and a half times as thick as normal sleepers. The hole would then be backfilled with dirt and rocks and rammed tightly so the plank was not moving. The back of the Shelton regulator clock would be screwed to this and the clock would be chocked as well. Obviously this or a Smeaton iron frame were not firm enough in the case of Dusky Bay, and Wales decided to set the astronomical regulator clock case up on a sawn off tree trunk (Figure 2).

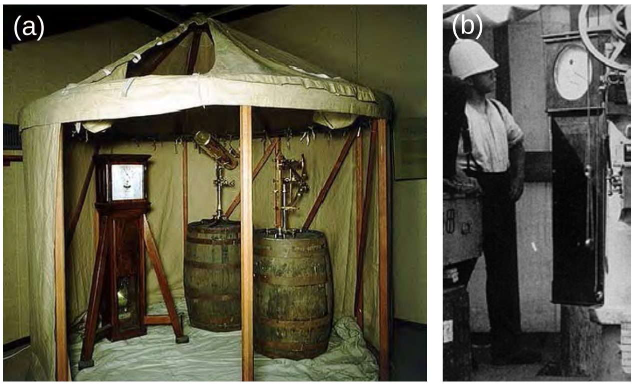

Figure 2: (a) Clock on the ground (National Maritime Museum, Greenwich) and (b) clock on sawn off tree trunk (Jimbour, Queensland, 1882).

He started his 16-day series of gravity observations on Monday, 5 April 1773. This long run of observations allowed the Shelton clock rate to be averaged for higher accuracy, by rating it against equal solar altitudes carried out daily with the quadrant. The gravity observations at Dusky Bay started about the same time as when the lost ship, the Adventure, arrived in Queen Charlotte Sound 800 km to the north of them, after having lost sight of Cook's ship two months earlier.

Wales summarised the Dusky Bay results of his 16-day observation run of gravity observations as follows (Figure 3):

> At Duskey Bay in New Zealand, latitude 45º 47' 5/12 S, longitude 166º 18' E., Clock B gained 4' 0.56" on sidereal time, from April 5th to 21st, 1773; and its mean vibrations were 1º 35' each way.

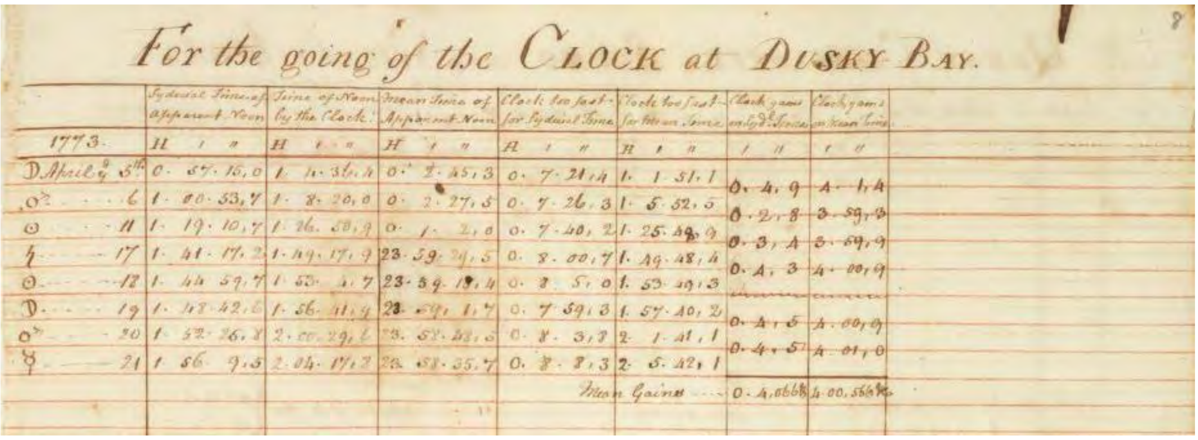

Figure 3: Clock rate summary.

The clock rate results had a standard deviation of the mean of about 1/3 of a second of time which is at the level of 4 parts per million of the length of a day. The amount of time the clock gains per day is a measure of the local gravity when considered together with the pendulum length and after allowing a correction resulting from the swing amplitude. Wales also listed his seconds pendulum calibration data from which the pendulum's reduced length can be determined:

> The clock B gained 5.03" a day on sidereal time from March 28th to April 1st, 1772, when fixed up at the Royal Observatory in Greenwich Park, to pieces of wood let into the wall of the Observatory; that is, in the manner whch the Transit Clock at that place is fixed up.

From Henry Kater's work we know the reduced length of the seconds pendulum beating 86,400 seconds in a mean solar day in Greenwich is 39.13858 inches in today's definition of the international inch of 25.4 mm. From the above calibration data we can see that Wales' pendulum, as set (the index of the bob was set at 13 on the nut, according to him), was slightly shorter so that it gained 5.03 seconds.

We can calculate the relevant length from this with the pendulum equation – this is what the calibration data allows us to do. From the pendulum equation we see that at a given value of the gravity, if we slightly change a pendulum's length it will swing at a different period, and the ratio of the squares of these two periods is the same as the ratio of the two slightly different pendulum lengths. This means Wales' pendulum length when set at 13 properly, was slightly too short as it gained against the astronomically determined value. The pendulum length is the square of 86,400/86,405.03 times Kater's "London pendulum length" of

39.13858 inches, resulting in 39.1340 inches. This was for Clock B carried on the Resolution, with Captain Cook and William Wales. Note that this is independent of the local gravity, which cancels in the comparison of the two periods. We get a relation between two lengths and two periods.

The full pendulum equation as in Giancoli (1988) is:

$$T = 2\pi \sqrt{\frac{L}{g}} \left(1 + \frac{1}{2^2} \sin^2 \frac{\theta_M}{2} + \frac{1}{2^2} \frac{3^2}{4^2} \sin^4 \frac{\theta_M}{2} + \cdots \right) \tag{1}$$

where θ_M is the "arc from the vertical" or half amplitude of the swing, L is the length of the pendulum in metres, g is the acceleration of gravity in m/s^2, and T is the period of a full swing and return of the pendulum in seconds.

However, Wales describes a mistake he made at Dusky, in his words: "as the difference corresponds nearly to that which would arise from a whole revolution of the nut which supports the ball of the pendulum". William Wales explains most of this in his introduction of the printed version of his and William Bayly's observations in Wales and Bayly (1777). Bosloper (2010) found this had also happened to William Dawes and he calculated the change to the pendulum length as 0.0258 inches for a full turn of the nut. This means that the effective length for the pendulum used by William Wales at Dusky Bay was 39.1340 minus 0.0258, i.e. 39.1082 inches. This allows us to calculate his absolute gravity measurement at Dusky Bay as all the other necessary variables are given.

In a similar way, we can accurately determine the pendulum length of Clock C of William Bayly's pendulum in Queen Charlotte Sound as mentioned earlier. It was only three tenthousandth of an inch shorter than its intended length, according to its calibration data as shown in Howse (1969) and Wales and Bayly (1777). So it was 39.1383 inches. This gives us the result of the month-long second absolute gravity determination in New Zealand, carried out by the gravimetry field party of the Adventure. The gravimetry here was carried out between 20 April and 20 May 1773, just after William Wales' gravimetry in Dusky Bay. The references to Clock B and C are also explained in Howse and Hutchinson (1969).

As a debrief later, William Wales described how the gravimeter was deployed with large intervals and spent the rest of the time in a damp and improper place in the ship. It should be stored in a more fit and proper place. This would be upright in the middle of the ship, which was to be lined with painted canvas overlain with thick baize. Often the pendulum spring would accumulate rust and even possibly bend at a different place which could affect the effective pendulum length. Thick baize might help with keeping moisture from accumulating.

4 ADVENTURES OF THE EARLY GRAVIMETRY FIELD PARTIES

In Rickard (2015) an interesting incident is described. During this second gravity determination, which lasted a full month, the party on the shore was met by a number of native New Zealander warriors who had been sent towards them. They had possibly become impatient by this long stay of the strangers in Queen Charlotte Sound and tried to intimidate them with a haka. James Burney, one of the crew of the Adventure describes it as follows in a May 1773 entry in his diary of the journey: "They all get in a row & jump & put themselves in many different strange attitudes, sometimes rolling their eyes about frightfully. One of

them speaks a number of short sentences, at every one of which they change their posture, keeping exact time and very regularly through all their motions…".

In Henry and Marra (1775) a description can be found of theft of survey material in December 1773 during the next gravity observation series. It appears based on the journal of John Marra, another member of the crew. This happened after the return of the Adventure from Tonga, a trip during which the lighter bark Adventure again lost sight of the heavier Resolution for more than a month. During a fierce northerly gale they last sighted each other, trying to round Cape Pallisier of the North Island. The lighter bark could not turn West in this gale without the risk of capsizing and had to return 300 km to the north to Tolago Bay twice to find refuge from the gale, before it could return south and enter the Cook Strait in a third attempt more than a month later. When they arrived in Queen Charlotte Sound, the Resolution had already left a week earlier, but Cook had left a message for them confirming that they had been there. This message made no mention of having encountered any hostilities.

The crew of the Adventure prepared for more observations. After the seventh day of the gravity observations, survey equipment had disappeared in the night of 13 December out of the astronomer William Wales' tent observatory. According to John Marra's journal, William Bayly was about to do observations when he saw that his quadrant had disappeared together with other equipment, and while he was mouthing off the guard he saw a native New Zealander sneaking out of the tent observatory. The astronomer fired a gun in his direction and wounded him but he got away into the bush. This alarmed the New Zealander's companions and they ran off into the bush as well, leaving their boat behind on the beach with the stolen goods. The wounding of the thief had severe consequences a few days later.

It rained the next day, but William Bayly did one more day of clock comparisons on 15 December 1773 so the retrieved quadrant appeared still to be in working order. Two days later everything was back on board in the afternoon for departure the next day. But a party of seven men who had been sent in a boat to gather some greens and wild celery from Grass Cove, had not returned yet. This included a midshipman, a carpenter's assistant and a first mate, possibly some of the crew who had helped the astronomer with his site preparation and such and were comfortable with being on shore in unknown and possibly hostile territory. At day break another party of seven was sent to look for them, among which was James Burney, the second lieutenant. His journal and John Mara's tells the story. It was not till five in the afternoon that they came upon some locals who fled from a canoe they were hauling onto a beach. In this canoe they found the first remains of their mates. This was followed by a battle with more New Zealanders which they encountered at arrival at Grass Cove, after finding more remains of four of their crew there, described as "heads, hearts and livers". They gathered the remains they could find and returned to their ship by midnight, unable to find the lost row boat. Tied in a hammock and cast overboard with "ballast and shot", the unfortunate men were later given a seaman's grave with the usual solemnity of the occasion. When the Adventure got clear of land, the clothes and effects of the lost men were sold before the mast as is a sea custom.

During his third journey, Captain Cook was told by the New Zealanders that the thief of the quadrant had succumbed to the wound inflicted by the astronomer's or guard's gun and this brought about the anger of the New Zealanders. These were the dangers of gravimetry field work in uncontrolled surroundings.

5 CONCLUDING REMARKS

John Harrison's isochronal temperature compensated gridiron pendulums have the same "reduced length" as later determined by Kater, because they have the same period. In hindsight, using Kater's value for the London isochronal pendulum, one can recalculate the results of Cook's absolute gravimetry to an accuracy of 10 mGals when using their pendulum calibration information, equalling accuracy levels not obtained again until the middle of the 20th century. This and their adventures leaves the author with tremendous respect for the gravimetric crews, astronomers and clockmakers of that period.

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National ePlan Working Group Update

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ABSTRACT

The Intergovernmental Committee on Surveying and Mapping (ICSM) has endorsed an ePlan standard, initially based on LandXML (LXML) as the national standard for digital lodgement of cadastral plans. As technology moves forward, ICSM understands that alternatives to this standard may need to be considered. The lodgement of digital survey data in an XML format will gradually replace TIFF/PDF as the file format for digital lodgement of plans in the ePlan portal. The LXML Model, for those jurisdictions that have adopted it (Victoria, Queensland and NSW), accommodates all the survey geometry, administrative and titling data to process a plan from lodgement and registration, through to the Digital Cadastral Database (DCDB) update. The ICSM publication 'ePlan Protocol LandXML Mapping' (available at http://www.icsm.gov.au/eplan/ePlan-Protocol-LandXML-Mapping-v2.1.4.pdf) defines every element within the LandXML schema. Jurisdictions will utilise the elements within the schema reflective of their respective requirements. The role of the ICSM ePlan Working Group has been to work together to implement LXML throughout all jurisdictions of Australia, New Zealand and Singapore. The role of the working group will change over time. It needs to consider the impact of technology advancements, the changing nature of international standards, and the changes in the regulation of land development and the impact of this on survey data. This presentation will provide an update to industry on the current state of play in the various jurisdictions that participate in the National ePlan Working Group, and the vision for the future role of the group.

KEYWORDS: *ePlan, LandXML, digital lodgement, standards, ICSM.*

Unlocking the Potential of UAS for Surveying and Mapping

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ABSTRACT

An Unmanned Aerial System (UAS) is a cutting-edge tool for surveying and mapping to supplement and extend conventional surveying technology in a number of ways. The UAS aerial photogrammetric survey capability has numerous applications, such as land surveying, mining surveying, construction monitoring, volume estimation and agriculture. Automatic and robust UAS photogrammetric image processing software packages require little professional surveying knowledge or expertise to produce impressive geospatial products, such as textured 3D point clouds, digital surface models and ortho-mosaic image maps. This implies that everyone can now execute a photogrammetric project. However, can they state the true accuracy of their UAS survey in the same manner that surveyors do? UAS surveying is a specialised engineering task, which requires surveying and geospatial professional expertise to plan the image collection task, efficiently conduct the aerial survey and evaluate the products' precision or accuracy. With this in mind, the School of Civil & Environmental Engineering at the University of New South Wales (UNSW) has acquired several survey-grade, fixed-wing and multi-rotary Unmanned Aerial Vehicles (UAVs) with cameras, obtained the RPA Operator's Certificate from CASA, and has been teaching UAS photogrammetric mapping in surveying and geospatial engineering subjects since 2014. This presentation will discuss UAS surveying project planning, UAS operations in Australian airspace, ground control point surveys, aerial image acquisition and processing, product accuracy evaluation, and introduce some UAS photogrammetry applications. In addition, it will be discussed how UAS technology is supporting teaching and research at UNSW.

KEYWORDS: *UAS, UAV, surveying, mapping, photogrammetry, geospatial.*

Deformation Survey of Newcastle Breakwalls using UAV Technology

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ABSTRACT

Monteath & Powys were approached by the Port of Newcastle to survey and monitor the rock armour surrounding the Nobbys and Stockton Breakwalls forming the entrance to Newcastle Harbour. Previously, the rock armour was monitored by visual inspection only and maintained with no recordable evidence of deformation within the rock walls. This was a costly exercise and posed a substantial risk of failure given the magnitude of the task. In order to monitor each of the thousands of rocks along each breakwall, we needed a method of capturing the data safely, efficiently and accurately. The idea of utilising an Unmanned Aerial Vehicle (UAV) for this purpose came from our experience in aerial surveys combined with our 3D laser scanning capability. Monteath & Powys have operated high-precision 3D laser scanning equipment for a number of years to report on deformation of structures compared to either previous surveys or design models. This paper outlines how we were able to combine the above-water and below-water data to create seamless Digital Terrain Models (DTMs) for comparison and deformation analysis over time. There were significant challenges in incorporating these technologies, but the resulting process allowed Monteath & Powys to provide a superior product to the client in a safe, efficient manner. This project was completed successfully in conjunction with the Port Authority NSW. It was entered into the 2015 Spatial Excellence awards, winning the Spatial Enablement category. This qualified the project for the 2015 Asia Pacific Spatial Excellence awards in Melbourne, which it also won. The success of this technology has led to several other projects including the survey capture of Eden Breakwater and the current Balmain Wharf to Birchgrove Wharf for Roads and Maritime Services.

KEYWORDS: *UAV, 3D modelling, laser scanning, point cloud, deformation monitoring.*

1 INTRODUCTION

The Port of Newcastle is the largest bulk shipping port on the east coast of Australia and the world's leading coal export port. It handles more than 25 different cargoes and 4,600 ship movements per year, all reliant on the protection offered by the breakwalls at Nobbys and Stockton. The breakwalls are constantly battered by the ocean with large swells causing movement in the rock armour, resulting in scouring and possible collapse of the breakwall structure. Previously, the Port of Newcastle would visually inspect the breakwalls after large storm events to check for deformation in the rock armour. Any areas which looked to have

moved would be rectified with additional material. This method was costly and unreliable, risking a critical asset.

Monteath & Powys were approached by the Port of Newcastle to assist in this process by use of our Terrestrial Laser Scanning (TLS) equipment. Once the scope of work was established, it was evident that TLS would prove costly and not provide full coverage of each individual rock from a single direction. Scan locations would only be available from the walkway on top of each breakwall and not from the rocks themselves or the ocean.

A second option raised was the use of Mobile Laser Scanning (MLS) equipment from a vessel. This methodology was considered a viable option, but the cost was prohibitive and data capture is limited by a fixed location on the vessel itself. By capturing the data in this way, sections of the rock wall are hidden from view and not included within the final model.

Having completed several aerial surveys with Unmanned Aerial Vehicles (UAVs), the idea of capturing the breakwalls with photogrammetry was raised. By capturing several passes with the UAV at different camera angles and locations, it was thought that a more complete capture of each rock could be achieved by minimising the shadowing which hampered the laser scanning options. Figure 1 shows the flight level methodology adopted, and Figure 2 illustrates the camera locations and overlap.

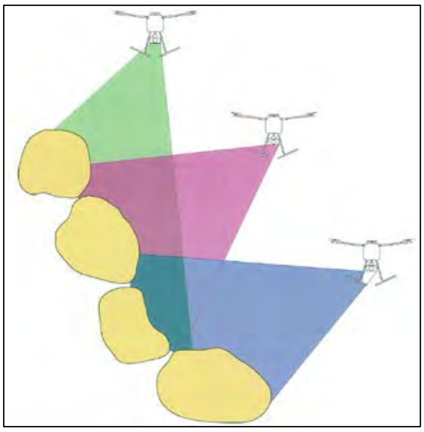

Figure 1: UAV photographic coverage in section view.

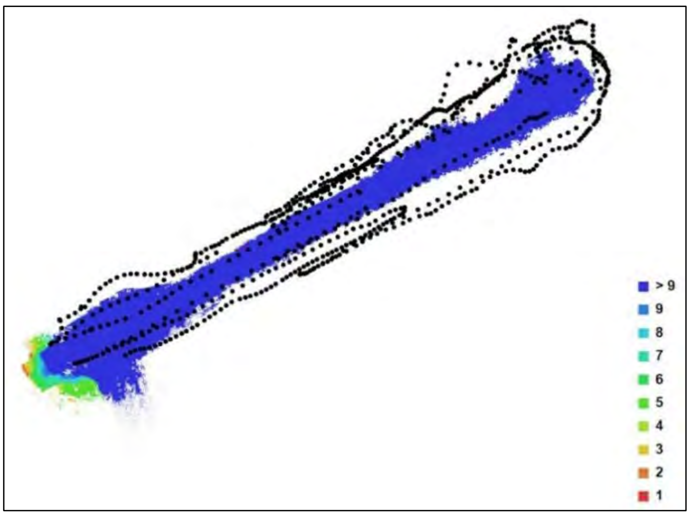

Figure 2: Camera positions and image overlap.

2 RESEARCH AND DEVELOPMENT

All photogrammetric survey work prior to this project had been completed with the camera angle perpendicular to the ground with survey control to orthorectify the unified image and create a Digital Terrain Model (DTM) accurate to approximately ±0.05 m. Along with Airsight Australia, we investigated the software limitations and were able to create a DTM from the combined aerial imagery despite the varying camera angles.

As with any survey process, we needed to prove beyond doubt that our method of survey was accurate and could be reliably replicated in future data captures. An extensive testing regime followed, which included multiple flights and 3D laser scanning of an easily accessible, sample section of breakwall. Using 3D Reshaper software (Hexagon, 2015), a DTM was created from each point cloud model. Each DTM was overlaid and checked for deformation within 3D Reshaper. The initial stages of testing identified some data problems, particularly in areas on the extremity of the survey. Errors up to 200 mm were found (Figure 3).

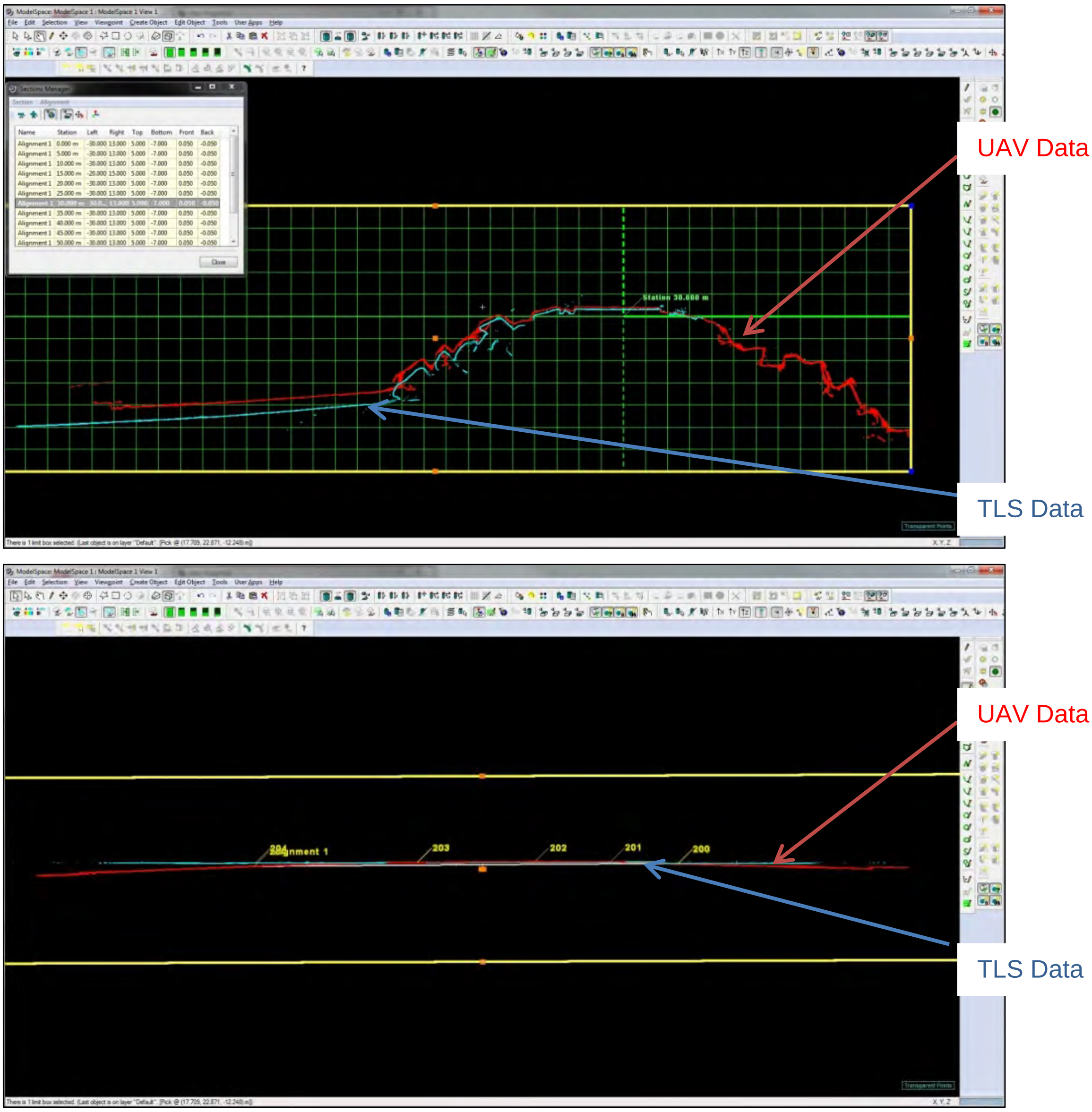

Figure 3: Initial errors between UAV and TLS data.

By reconfiguring the survey control to the extremities of the capture area and widening the linear control network, it was possible to reduce the DTM from the UAV with a higher level of confidence. When overlaid with the laser scanning data, the error between the two sets of data was found to be well within the ±0.05 m considered acceptable for this task. The DTM deformation plot between TLS and UAV data can be seen in Figure 4. With great confidence in our field capture technique, this methodology was presented to our client as a viable option for the ongoing monitoring and reporting of the breakwall structures.

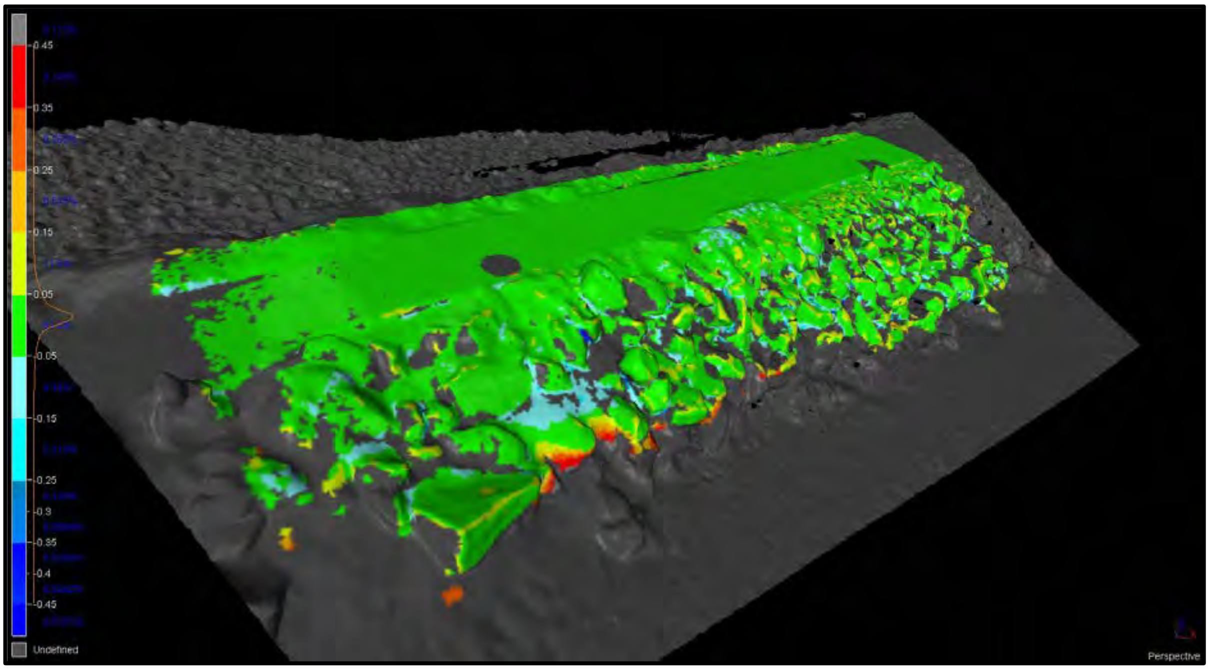

Figure 4: Colour deviation plot showing all areas within ±0.05 m coloured green.

3 FIELD CAPTURE

3.1 Initial Baseline Survey Capture (2014)

As part of the project deliverables, Monteath & Powys were asked to incorporate bathymetric Light Detection and Ranging (LiDAR) data from the Port of Newcastle's survey vessel. The data is captured on high tide to ensure maximum coverage is achieved. Conversely, Monteath & Powys were to capture photogrammetric data on an extreme low tide within 0.3 m of the Lowest Astronomical Tide (LAT). This occurs approximately once a month for a period of around three days. It is also best to capture the data with minimal swell to limit the amount of water splash in the image capture. Extreme wind conditions can also affect the flight of the UAV. In order to achieve accurate flight paths and image capture, the UAV can only fly in wind less than 35 km/hr.

Newcastle being coastal and exposed to the elements, the wind speed is often over 35 km/hr, particularly in the afternoons. With this knowledge it was decided the proposed capture dates should be aligned to the extreme low tides falling in the morning periods between 7.00 am and 10.00 am. This results in a small window of opportunity to capture each breakwall structure. Other events such as ship movements in and out of the harbour as well as defence exercises from the nearby RAAF base at Williamtown also caused delays in the field capture.

Once a suitable date was set, survey control was placed on Map Grid of Australia (MGA) coordinates and Australian Height Datum (AHD) levels (heights) using the Real Time Kinematic (RTK) Global Navigation Satellite System (GNSS) technique in locations chosen specifically to encompass as much of the capture area as possible. Survey control was placed using CORSnet-NSW (Janssen et al., 2016; DFSI Spatial Services, 2017) with two readings taken on each station using different initialisations. The accuracy of the control must be within ±50 mm, which was achieved easily using this this methodology. Marks placed were a 200 mm wide cross painted on the ground in locations visible from the air. Secondary control marks were also placed as wide on the wall as possible and on the beach to confirm results.

Flight paths were chosen at specific offsets from the breakwall centreline running parallel to the centreline and each other. This was designed from our preliminary research and development results to maximise the coverage of each individual rock and minimise shadowing. A diagram showing the flight paths can be seen in Figure 5. Flights were completed over a period of two days with minor interruptions caused by ship movements, wind and interested members of the public.

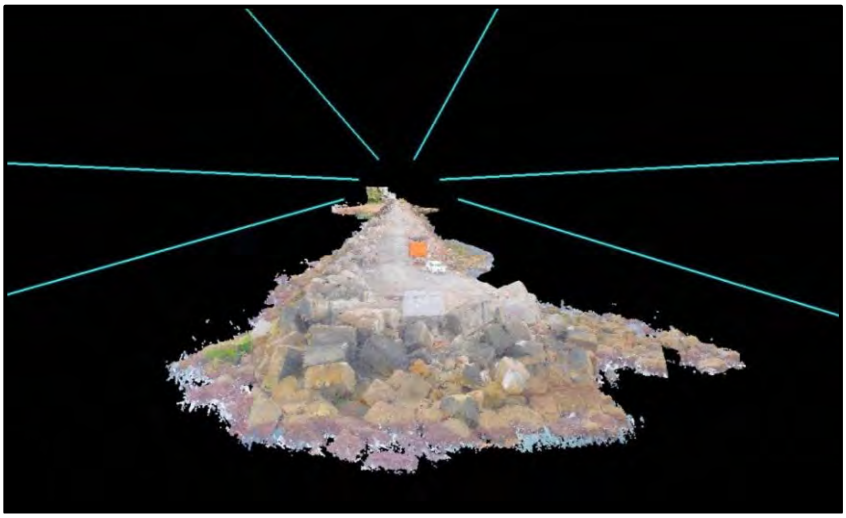

Figure 5: Flight paths used to capture the breakwall.

3.2 Secondary Field Data Capture (2015)

After a period of 6 months and several large storm events, Monteath & Powys were engaged to perform a secondary survey of the breakwall being the first comparison to the baseline dataset. Field capture was carried out in the same manner as the initial data capture using the same survey control network. As with any monitoring survey, keeping the methodology as similar as possible reduces the variables and potential for error in the datasets. The Port of Newcastle also had a secondary field capture of the bathymetry, allowing a full comparison of the above and below water rock armour to take place.

4 OFFICE COMPUTATIONS

4.1 Creation of DTMs

With two full sets of data, it was now possible to complete the comparison and report on deformation over time. This presented many challenges creating complex meshes of

thousands of rock structures including overhangs. Most software packages create DTMs by triangulation in 'plan view' and will not allow for triangulation under another triangulated surface. A diagram of a triangulated mesh from a standard surveying software package can be seen in Figure 6.

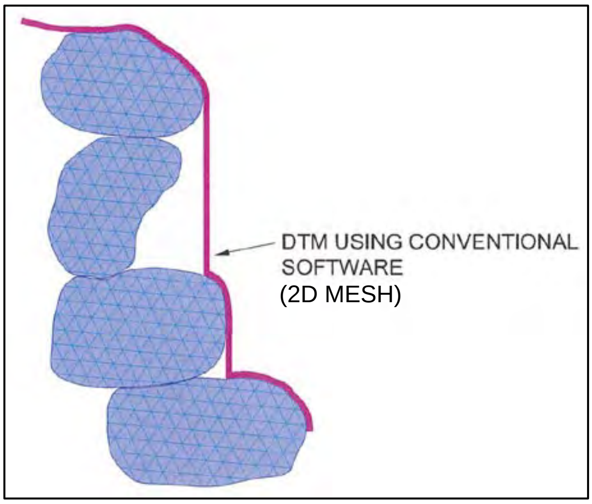

Figure 6: Typical rock structure section view using a conventional 2D mesh.

By using 3D Reshaper software (Hexagon, 2015) and our knowledge of complex meshes, it was possible to accurately model the entire visible surface of each individual rock including the underside in some instances. The complex mesh can exist above and below itself where the standard mesh cannot (Figure 7). Only a specialised few processing programs can perform complex meshing. The 3D Reshaper software was not cheap ($13,000) but well priced for functionality when compared to more expensive packages on the market.

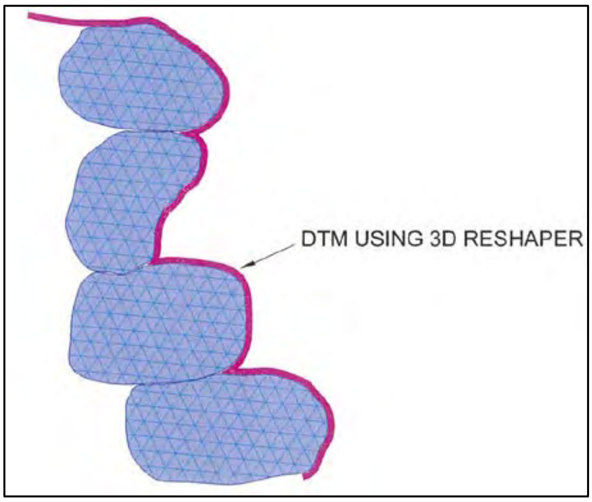

Figure 7: Typical rock structure section view using complex meshing via 3D Reshaper.

4.2 Data Comparison

A significant challenge in this project was the comparison of two large datasets. With over 65 million points per dataset, this may seem like a lot of data but in the laser scanning world this is a relatively small point cloud. The challenge was creating an accurate surface that could be compared to another accurate surface in a way that could be easily reported on and minimised false indication of movement between the two datasets.

Colour deviation mapping between two surfaces was deemed to be the most suitable reporting method for the task. By calculating a colour-coded map between the two surfaces, areas on both breakwalls that showed significant movement could be easily identified by means of a 'hot spot' or areas of a different colour (Figure 8).

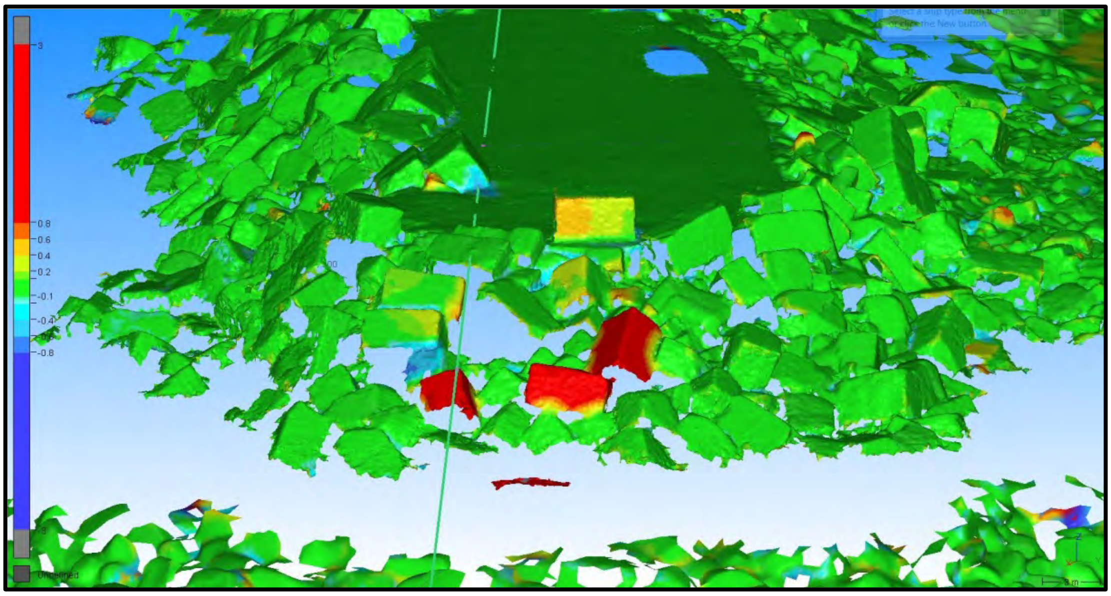

Figure 8: Example of colour deviation mapping on the southern breakwater. Positive values on the scale show addition to base surface, while negative values show removal of base surface.

The base surface in this case was created from the 2014 survey (see section 3.1) and compared to the surface created from the 2015 survey (see section 3.2). The perpendicular or normal distance between surfaces is then used to colour the nominated surface as shown in Figure 9. A positive value indicates a growth or addition to the surface, while a negative value indicates a retreating or removal from the base surface.

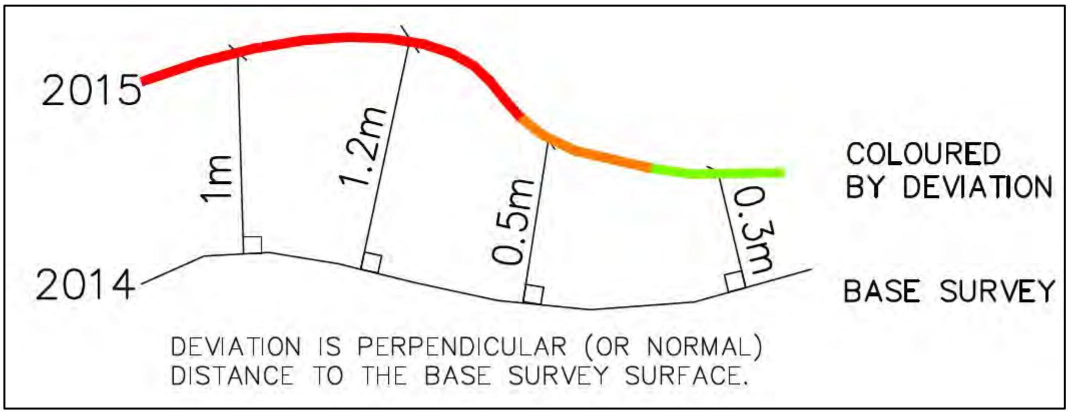

Figure 9: Colour deviation calculation method.

Before colour deviation mapping can take place, the two DTMs to be compared need to be created from the point clouds. The ability to create accurate surfaces that wrap around all the ins and outs, overhangs, etc. of this complicated structure accurately is the crux of the project. The majority of software packages are unable to compute a meshed surface that can exist above or below itself. This type of mesh is called a 'complex mesh' and was essential in this project.

By varying the angles of the camera position on the UAV, it was possible to maximise the point cloud coverage of the structure above water. Areas of sparse data or no data (i.e. shadows) still existed in the point cloud data. It was important to exclude these areas from being meshed to minimise false hot spots in the colour deviation mapping.

By using the meshing parameters to leave holes in the mesh when insufficient data was present, we were able to report or compare only true surfaces against true surfaces. While a water tight mesh usually looks more impressive, the triangulation across areas with little data could be different each time the survey is performed, creating false hot spots (Figure 10).

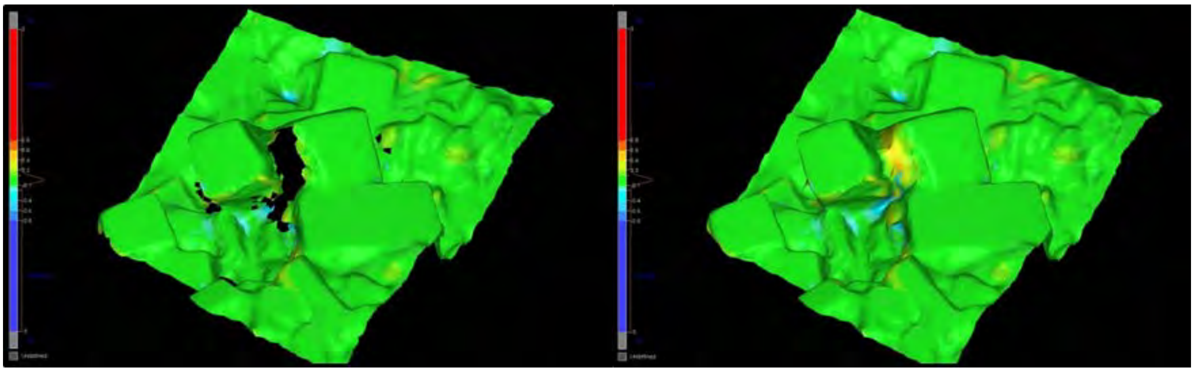

Figure 10: Comparison of the result between a mesh with holes (left) and a water tight mesh (right). In the latter case, a false 'hot spot' is created between two rocks shown.

The ability to eliminate false hot spots saves a significant amount of time in the next step of the process, matching hot spots with captured aerial images to identify possible causes for the hot spot and creation of a simple report. Each area of potential change is examined in both sets of images and reported on. A view is set up in 3D Reshaper so that the hot spot is easily identified in the digital data and referenced to the report. False hot spots would greatly increase an already lengthy process, so for efficiency and accuracy it is crucial to avoid them. Tight trimming of the UAV point cloud at the water line is also completed to minimise false identification. Photogrammetry creates points on surfaces visible in low lying water, which needs to be removed as these are usually incorrect.

5 PRESENTATION OF DATA

The results obtained showed several areas of change between the two surveys, and a simple reporting style needed to be created to concisely present the client with the location and magnitude of each occurrence. Each area of deformation from the UAV data highlighted by the software package was cross referenced to the images in that particular area. This allowed us to clearly show the movement in both a virtual graphic and photographic image. An example page of the full report is provided in Figure 11.

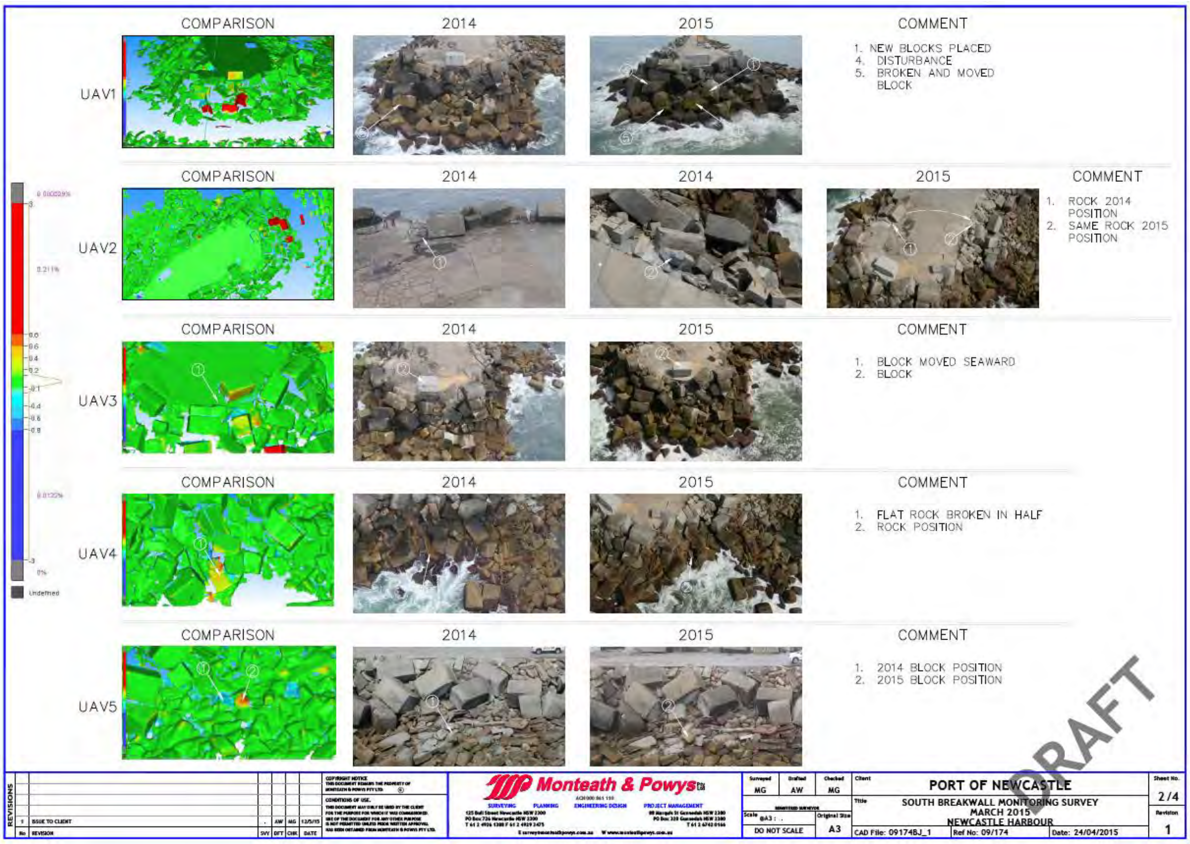

Figure 11: Example page from the report showing areas of detected deformation.

6 CONCLUDING REMARKS

This project has been incredibly successful for Monteath & Powys, allowing us to provide a service to several of our clients that was not previously possible. By combining these technologies, our clients have benefited from several aspects of the deliverables as well as promoting Monteath & Powys and the surveying industry. By utilising UAVs to produce a comprehensive point cloud of the rock wall structure, we were able to create a full 3D mesh which could be directly compared to the previous surveys. This was possible through our intimate knowledge of 3D data and specialised software packages acquired from our laser scanning experience.

As a promotional tool for both Monteath & Powys and the broader surveying industry, a cinematographer was engaged to create a 3-minute video to briefly outline the task and results we were able to achieve (Figure 12). This video was placed on YouTube and has so far received over 1,000 views (Monteath & Powys, 2014).

The response to the video has been outstanding, and we believe material such as this contributes significantly to the growth and maturity of the industry. By adapting to new technology and promoting ourselves and the industry in a professional manner, we can continue to enhance the public’s perception of the surveying and spatial information industry as a whole.

Figure 12: Screenshot of YouTube video showing the breakwall (Monteath & Powys, 2014).

ACKNOWLEDGEMENTS

Special mention must be made to Airsight Australia who we have formed a close relationship with to operate the UAV. By using Airsight Australia as the chief operating pilot, we have access to a range of UAVs for different purposes. We now have internal staff members trained in the operation of the UAV camera and form a combined team with the pilot from Airsight Australia. This relationship has allowed us to provide clients with a full range of UAV options, and not just a single limited-use UAV.

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Combining Multi-Source Point Cloud Data

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ABSTRACT

Point clouds are more readily available than ever before with literally hundreds of sensors and software packages delivering to our desktop hundreds of millions of points. This presentation examines how we can look to merge Light Detection and Ranging (LiDAR) data from terrestrial and Unmanned Aerial Vehicles (UAVs) to achieve a combined dataset with coverage not possible from a single sensor. When merging this data, it is important to know how to control and verify the quality of the model and the source of the data. This presentation helps understand the limitations of each scanning capture method for project planning purposes.

KEYWORDS: *Point cloud, LiDAR, scanning, UAV, UAS.*

Spatial Services Initiatives to Improve the Preservation of Survey Infrastructure

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ABSTRACT

Survey marks are protected under Section 24 of the Surveying and Spatial Information Act, and significant penalties are applicable if the proper procedures are not followed for their removal and replacement. While survey marks underpin the definition of cadastral boundaries, they are not recognised as critical or significant like other pieces of infrastructure such as telephone, water or gas. Historically, survey marks have been destroyed in large numbers with no consequences, either through ignorance or blatant disregard for their importance. The Office of the Surveyor General recognised that work needed to be done to rectify this situation and has been working on a number of initiatives, including the remake of Surveyor General's Direction No.11 (Preservation of Survey Infrastructure), signing the collaborative agreement with Roads and Maritime Services (RMS), and the Survey Marks App. This presentation outlines some of these changes and the innovations being implemented by Spatial Services staff to manage and mitigate the organisation's risk and ensure the integrity of the state control survey network and cadastre are maintained.

KEYWORDS: *Preservation of survey infrastructure, SGD11, POSI, Survey Marks App.*

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City of Sydney Permanent Survey Marks: Historical Context and Management into the Future

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ABSTRACT

The City of Sydney is in a unique position with regards to boundary surveying in NSW. The Sydney Corporation Act 1879 effectively aligned the streets based on kerbs as laid, the geometry of which is maintained today by a network of permanent survey marks which also provide Map Grid of Australia (MGA) and Australian Height Datum (AHD) control for surveyors. Unfortunately, the network has been severely impacted by reconstruction works over the last 30 years but, with the cooperation of DFSI Spatial Services, the network has been restored and indeed enhanced. Local Government has also recently gone through a significant change in reporting requirements, particularly around infrastructure asset management. Internally we have seen this as an opportunity to embed permanent survey marks into the mainstream of infrastructure asset management, like a road or a drain, with an acknowledged financial and community value. The replacement value of the City's permanent survey mark network alone is $3.5 million. This recognition gives the impetus to gain management support to:

- *Complete periodic inspections of all marks in the Local Government Area (LGA).*
- *Annually review budget for mark maintenance and restoration.*
- *Improve mark retention through development application conditions, sympathetic design and contractor education.*
- *Participate in Dial Before You Dig.*

This paper begins with a brief history of the permanent survey mark network and the related placement, replacement and coordination activities. It then outlines the development of a register and mobile capability in our corporate asset management system to enable effective permanent survey mark asset management. This includes:

- *Creation, modification and historicising assets.*
- *Scheduling and completing periodic inspections.*
- *Raising and completing maintenance jobs and tracking progress.*
- *Updating information in the City's corporate Geographic Information System (GIS).*

The City of Sydney has recognised the importance of the permanent survey mark network and is utilising core corporate technologies and processes to better manage and protect this critical infrastructure.

KEYWORDS: *Permanent survey marks, survey infrastructure, asset management.*

1 INTRODUCTION

The City of Sydney (the City) was established in 1842 and since that time has grown and contracted as state governments have redrawn boundaries. Today, the City covers approximately 26.15 km^2 (Figure 1), has a population of over 200,000 and has approximately 1.2 million people in the area daily.

Figure 1: The City of Sydney boundaries.

As the City grew in the 1800s, better regulation of the property boundaries was required which led to the establishment of the Sydney Corporation Act 1879. This Act effectively aligned the streets and placed a statutory obligation on the City to establish and maintain a network of permanent survey marks to preserve the alignments. This obligation has presented many challenges to the City's surveyors, particularly the often destructive impact of reconstruction activities including the 2000 Olympic Games footway reconstruction.

The City recognised the critical importance of the permanent survey mark network to the cadastral infrastructure of NSW, which has been emphasised by various sources (e.g. de Belin, 2012; Ward, 2014; DFSI Spatial Services, 2015). Together with Spatial Services, a unit of the NSW Department of Finance, Services & Innovation (DFSI), the City developed a comprehensive program to consolidate and (where necessary) re-establish the network. The coordination of the network followed and finally a City Alignment Recovery Plan was registered at DFSI Spatial Services.

The City's surveyors have reinforced the control network by developing processes and documentation around the preservation of the survey infrastructure that focus on education and incorporation of standard conditions on developments and construction activities. Importantly, survey infrastructure is now considered as a City asset in the same way as a section of road, tree or drainage line. This embeds survey infrastructure in the mainstream of infrastructure activity where an understanding of replacement value, maintenance cost and long-term renewal planning is required in order to gain budgetary support to ongoing funding.

To facilitate the capture of the necessary data, the City has developed an asset register, inspection regime and defect and job tracking process for permanent survey marks using the Corporate Asset Management System (CAMS) provided by Confirm (Pitney Bowes Software, 2017). The solution uses desktop and mobile technology, interfacing to the corporate Geographic Information System (GIS) to allow City staff to view and track activity against permanent survey marks in the same way as other infrastructure.

This paper outlines the historical significance of the permanent survey mark network and the related placement, replacement and coordination activities in order to consolidate the survey infrastructure. It then describes the use of standard corporate applications to enhance both the protection and management of this critical infrastructure.

2 CITY OF SYDNEY SURVEY INFRASTRUCTURE HISTORY

Although the City of Sydney as we know it today came into being in 1842, land development of the area had been going on for some 50 years or so before this date. With respect to survey infrastructure, particularly within the CBD, the first significant legislation of note is Act 4 Wm IV No. 7, which (amongst other things) provided for the appointment of a Town Surveyor or Surveyors whose duty it was to approve and supervise footway improvements, the opening of footways and carriageways and several other items relating to development in and about roads. This legislation also required the Surveyor General to survey and mark the town, marking with posts the corners and intersections of the streets (Cadogan, 1997).

In 1835 came new legislation, Act 5 Wm IV No. 20, “an Act for better regulating the alignment of streets in the Town of Sydney”. This Act makes reference to the alignment posts placed under Act 4 Wm IV No. 7 and made it unlawful to build within 12’ of the curb-stone of the footway. It also required the Surveyor General to “lay before the Governor a plan of the carriageways and footways set out as part of the alignment process”.

1842 saw the first of several Incorporation Acts before the Sydney Corporation Act of 1879. The 1879 Act repealed all previous road and alignment legislation and gave Council full control over the streets within its boundaries. Importantly, it also declared that the City’s sandstone kerbs in existence at the time were monuments, meaning that numerous streets became defined by measurement from the kerb line.

As kerbs were replaced and alignment posts lost due to age and attrition, the recovery of alignments became increasingly troublesome. In the late 1920s, City Surveyor Victor Waine floated the idea of defining alignments with a system of permanent survey marks rather than from kerbs. The idea faced significant opposition, partly due to the Great Depression limiting works to essential maintenance and partly from people who were of the opinion that the alignments could only ever be defined from the monuments (Cadogan, 1997).

Eventually reason prevailed, and City of Sydney PM 1 (a brass tack set into a lead plug in a concrete foundation) was placed on Parramatta Road “3’ south of the northern kerb line of Parramatta Road on City Boundary at Orphan School Creek” (Figure 2).

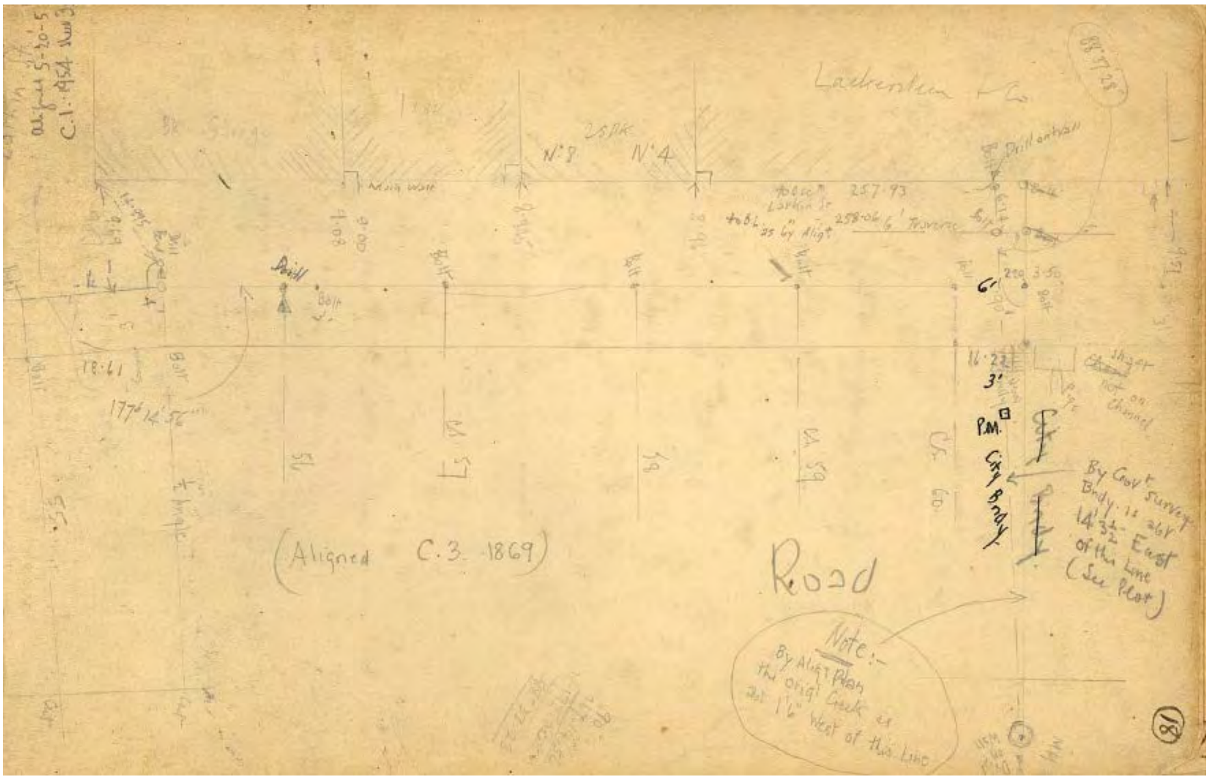

Figure 2: Field Book 160 Fol. 18, showing the marking of City PM 1.

In 1986, City Surveyor Thomas Clarke replaced the original mark with a brass bolt and renumbered it PM56546 (Figure 3) in order to enter the mark into the Survey Control Information Management System (SCIMS – see Kinlyside, 2013). Incidentally, as of June 2015, the mark was still in situ (although Orphan School Creek is not), despite the best efforts of road resurfacing crews (note the gouge marks on the lower edge of the box in Figure 3).

Figure 3: PM56546, formerly City PM 1.

Of course, within the City there are now a significant number of Permanent Marks (PMs), State Survey Marks (SSMs) and Cadastral Reference marks (CRs) placed by various people and organisations. At present, there are approximately 2,000 'live' SCIMS marks, around 1,000 City of Sydney alignment PMs not yet in SCIMS and an (as yet) unknown number of CRs, although we are in early discussions with DFSI Spatial Services about the cadastral back capture as a way of identifying and preserving this information as well.

3 CITY ALIGNMENT RECOVERY PLAN

After Sydney won the right to host the 2000 Olympic Games, then Lord Mayor Frank Sartor initiated a major City beautification program, including the widening of footways and major pavement upgrades. As a result of these works, numerous permanent survey marks were lost.

In the years following the Olympic Games, the City's survey team under Mr Peter Godfrey Harris (OAM) redefined the alignments and replaced around 650 missing permanent survey marks (Cadogan, 2012). This involved relocating the original position of each mark from field notes, followed by construction of new permanent survey marks, many of which now lie in the widened footway, improving accessibility and (hopefully) lifespan – although recent works in the City are again placing the alignment marks under threat.

The new marks helped re-establish the alignments of many City streets, however they still lacked high-accuracy Map Grid of Australia (MGA) and Australian Height Datum (AHD) values to further increase their utility. In a collaborative effort between the City and DFSI Spatial Services, the entire network of CBD permanent survey marks was levelled and traversed over the period of 10 years – a great example of governmental cooperation. DFSI Spatial Services surveyors collated all of the data that was collected and carried out a least squares network adjustment that resulted in MGA coordinates and AHD values being determined for a large number of marks within the network.

It was also decided that the information should be collated and put on public record as a way of making the alignment information easily accessible. The end result of all these efforts is a 71 page Plan of Survey Information, DP1196090, showing the network of marks and their relationship to the boundaries of the City's streets and the 1879 kerb lines (Figures 4-7). The plan took four years to draft and was lodged at DFSI Spatial Services on 30 April 2014.

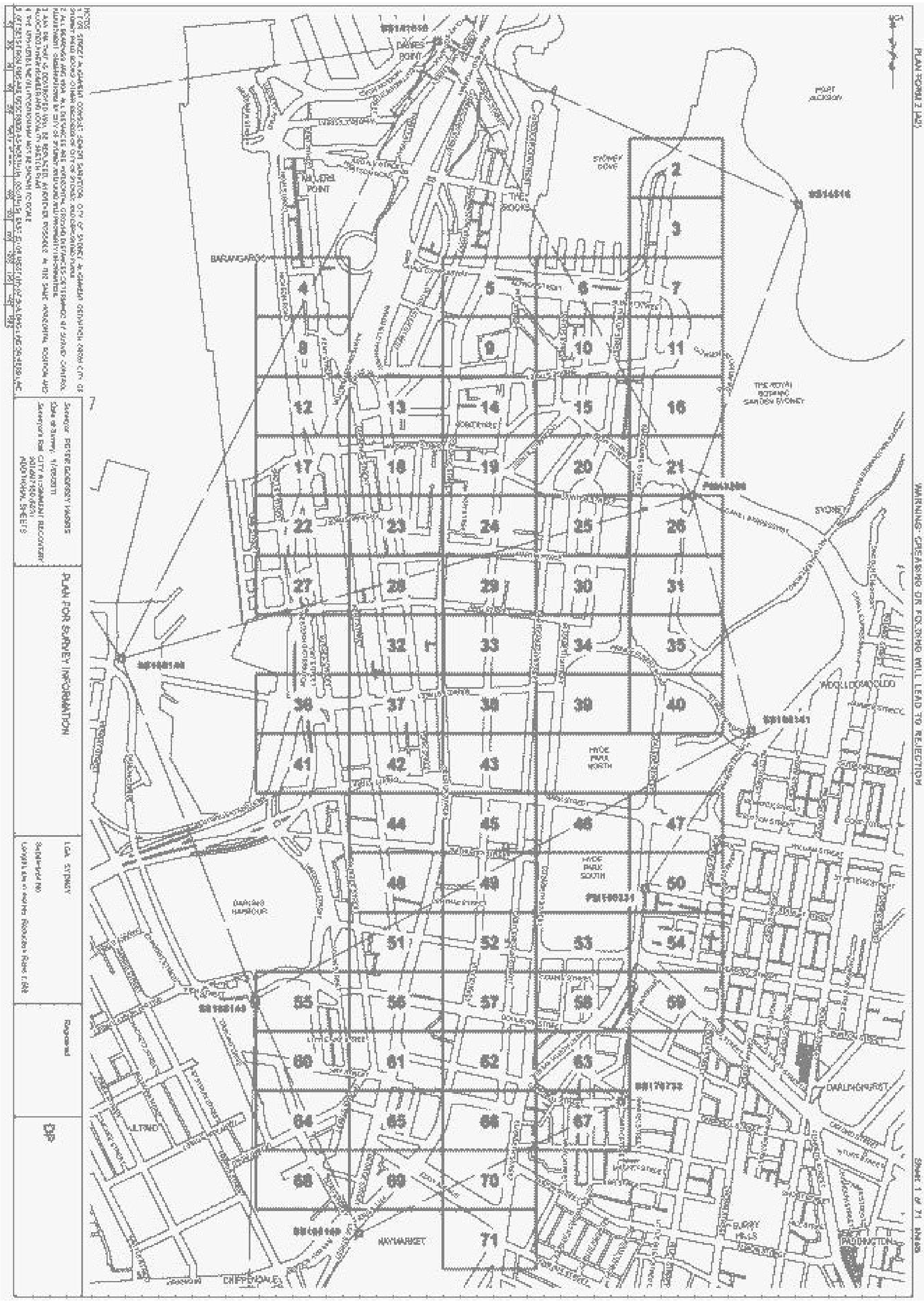

Figure 4: Sheet 1 of DP1196090.

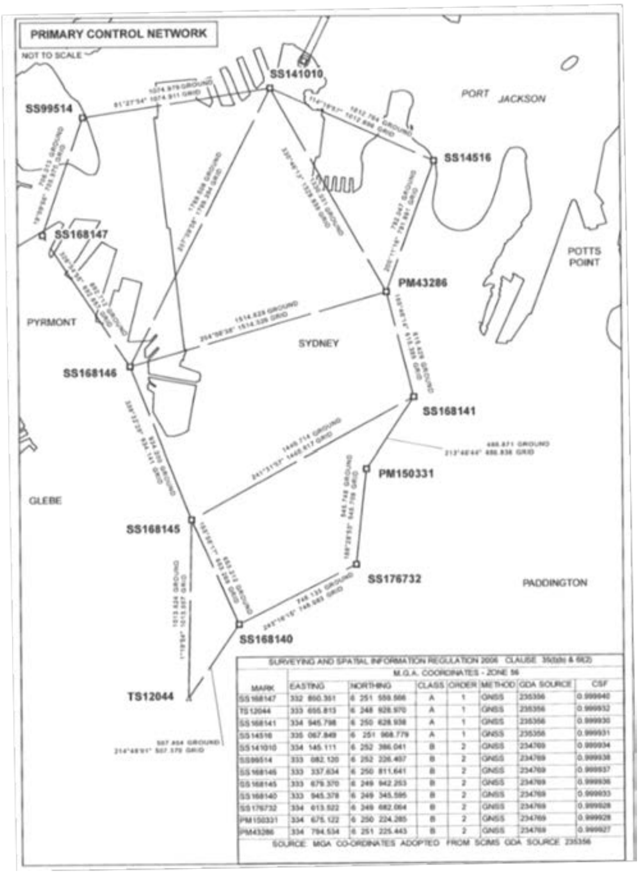

Figure 5: The control network for DP1196090.

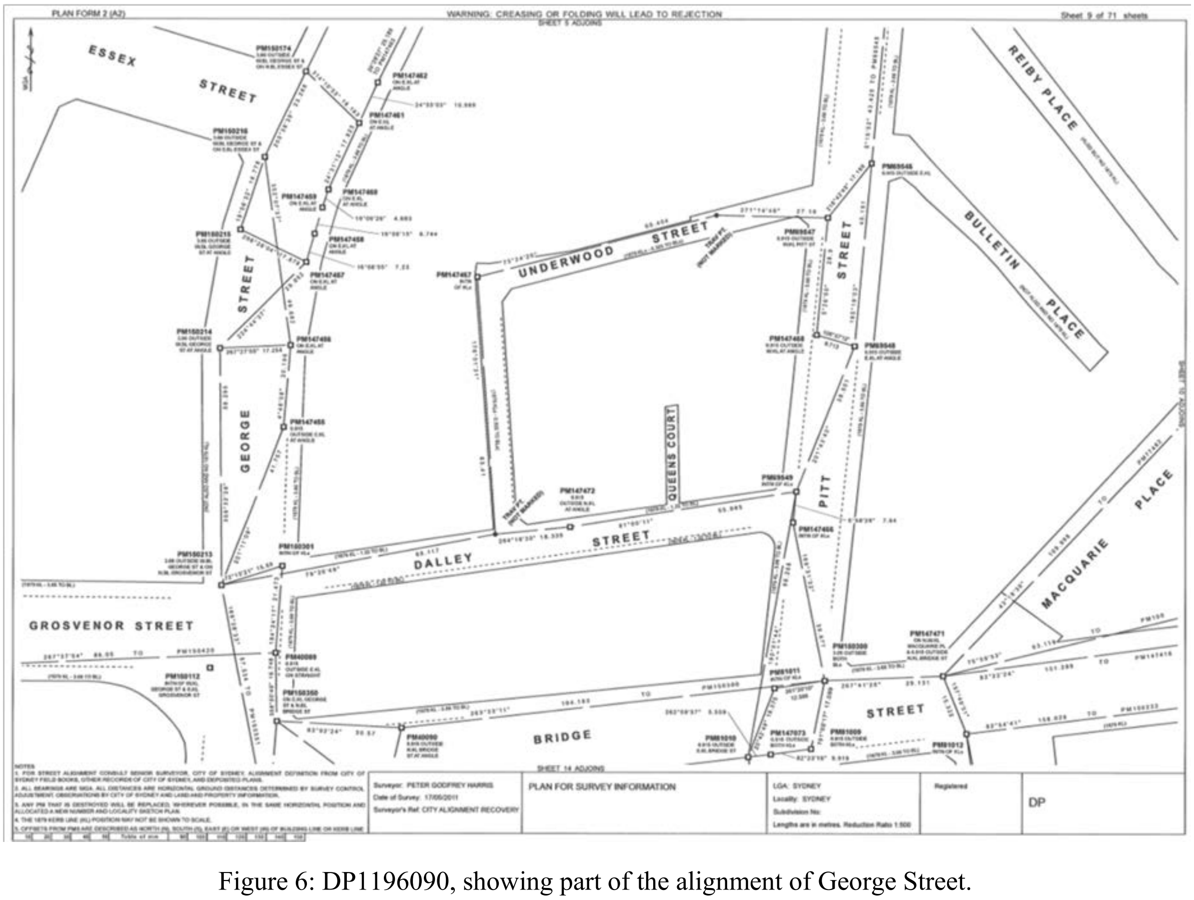

Figure 6: DP1196090, showing part of the alignment of George Street.

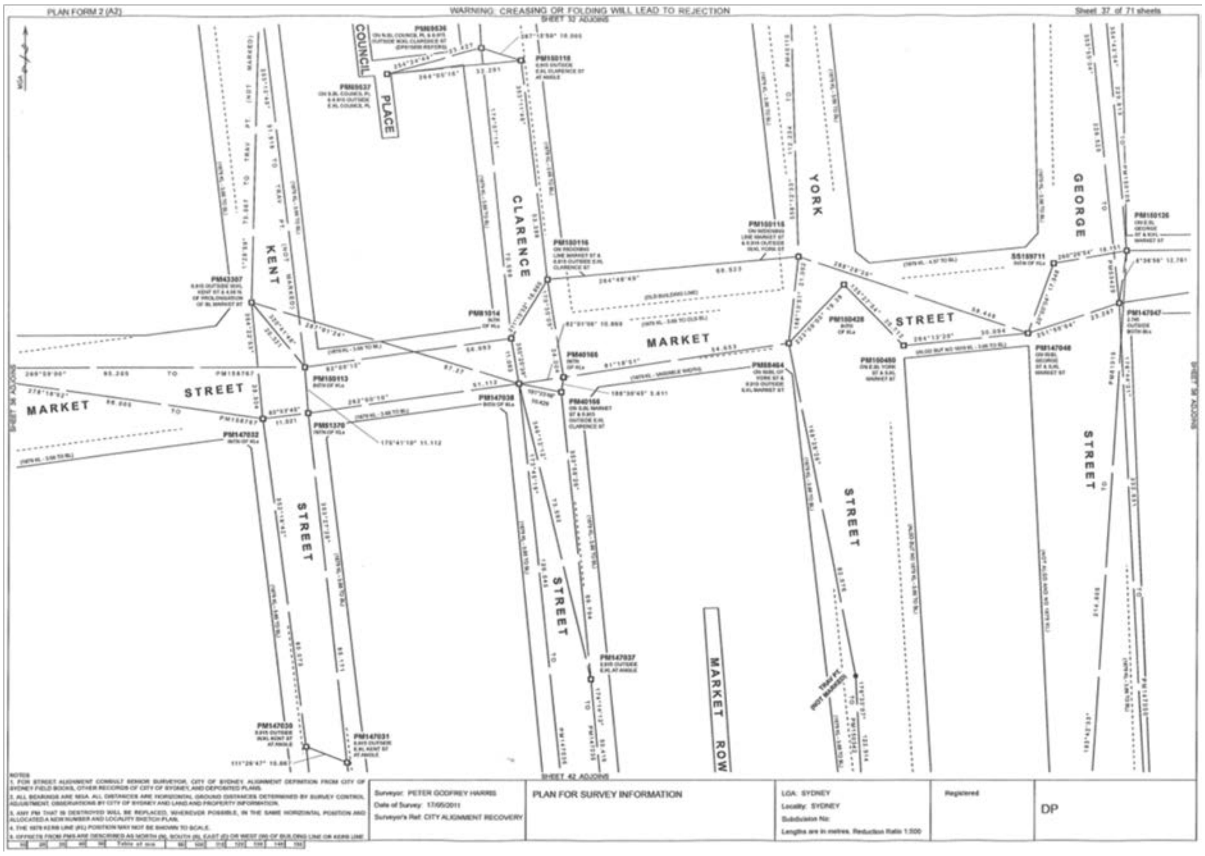

Figure 7: DP1196090, showing road widening in Market Street, with reference to 1879 kerb lines.

4 PRESERVATION OF SURVEY INFRASTRUCTURE & DIAL BEFORE YOU DIG

In a quantum leap of thinking about management of survey infrastructure, the City has within the past 3 years identified survey infrastructure as an 'asset' that should be managed and cared for as any other asset. This has involved a three-pronged approach:

1. Education – Through better education (of internal teams as well as construction contractors and the general public) it is believed that we can slow the attrition of survey infrastructure or at least ensure that the information is adequately preserved or replaced.
2. Funding – Recognising that marks will suffer attrition as well as wear and tear and hence allocating budget and resources to maintain and replace marks as needed.
3. Programmed asset inspections – Through regular inspections of mark functionality, we can identify problem areas, schedule maintenance or replacement works, and maintain and eventually improve network functionality.

To this end, the City has a number of initiatives currently under way:

- Standard charge for the replacement of survey infrastructure that can be levied on private developers, other government bodies and on our own internal teams.
- Conditions placed on Development Applications (DAs) relating to the preservation of survey infrastructure.
- Meet with and present regularly to internal teams and contractors about the importance of survey infrastructure to encourage sympathetic design.
- Seek funding each year for renewal and replacement, in keeping with asset management principles.

- Instated a rolling program of asset condition assessments, with the aim of ensuring that each known mark is inspected at least once every 5 years.
- Uploaded the permanent survey marks into Dial Before You Dig (DBYD) with information on the end user's responsibility under the Surveying and Spatial Information Act and information about how to proceed with survey infrastructure.
- Added the DBYD response as a condition of obtaining a Road Opening Permit, which will allow us to monitor works and recoup costs if and when marks are destroyed.

The experience with DBYD has been both challenging and encouraging. Initially, the manual processing of enquiries (i.e. each one was read, an appropriate map selected and individual email responses issued) proved problematic. We were receiving around 150 enquiries per week and each one was taking an average of 5 minutes to process, meaning we were spending 12.5 hours each week just on this task!

Armed with this information and the 'carrot' that other asset classes could be added for no additional cost, we successfully applied for funding for an automated reply service. Our DBYD service now has the City's survey, stormwater and electrical infrastructure uploaded and, as of January 2017, has handled in excess of 25,000 enquiries over 19 months.

Anecdotally, we believe that our initiatives are working. Certainly we have received plenty of enquiries and feedback from the public regarding DBYD searches, DA conditions and other survey infrastructure matters. Changes are also noticeable in the field, e.g. service trenches diverted, sections of kerb and gutter with marks in situ being retained, and much closer cooperation with our construction crews.

5 ASSET MANAGEMENT AT THE CITY

An asset is defined as "a resource controlled by a Council as a result of past events and from which future economic benefits are expected to flow to the Council (NSW Government, 2013). The term 'asset management' is defined in the City's Asset Management Strategy 2011-2021 as "the combination of management, financial, economic, and engineering and other practices applied to physical assets with the objective of providing the required level of service in the most cost effective manner" (City of Sydney, 2012). Asset management is a 'whole of life' approach that includes planning, purchase, operation, maintenance and disposal of assets.

The City of Sydney is responsible for some $8.8 billion in infrastructure, land, property, plant and equipment and spends about $250 million on maintenance, renewal, upgrade and expansion of critical infrastructure per annum. The replacement value of the City's permanent survey mark network is $3.5 million. The introduction of the Integrated Planning and Reporting legislation in 2009 placed a much greater responsibility on Councils to account for and plan for all existing assets under their control. The Integrated Planning and Reporting framework is shown in Figure 8.

The Resourcing Strategy supports the supports the framework and includes the Asset Management Plan where the obligations of good governance are detailed. Figure 9 shows the Resourcing Strategy framework at the City together with the supporting asset management policies and strategies. All asset management activity at the City is overseen by the representative Asset Management Program Control Group.

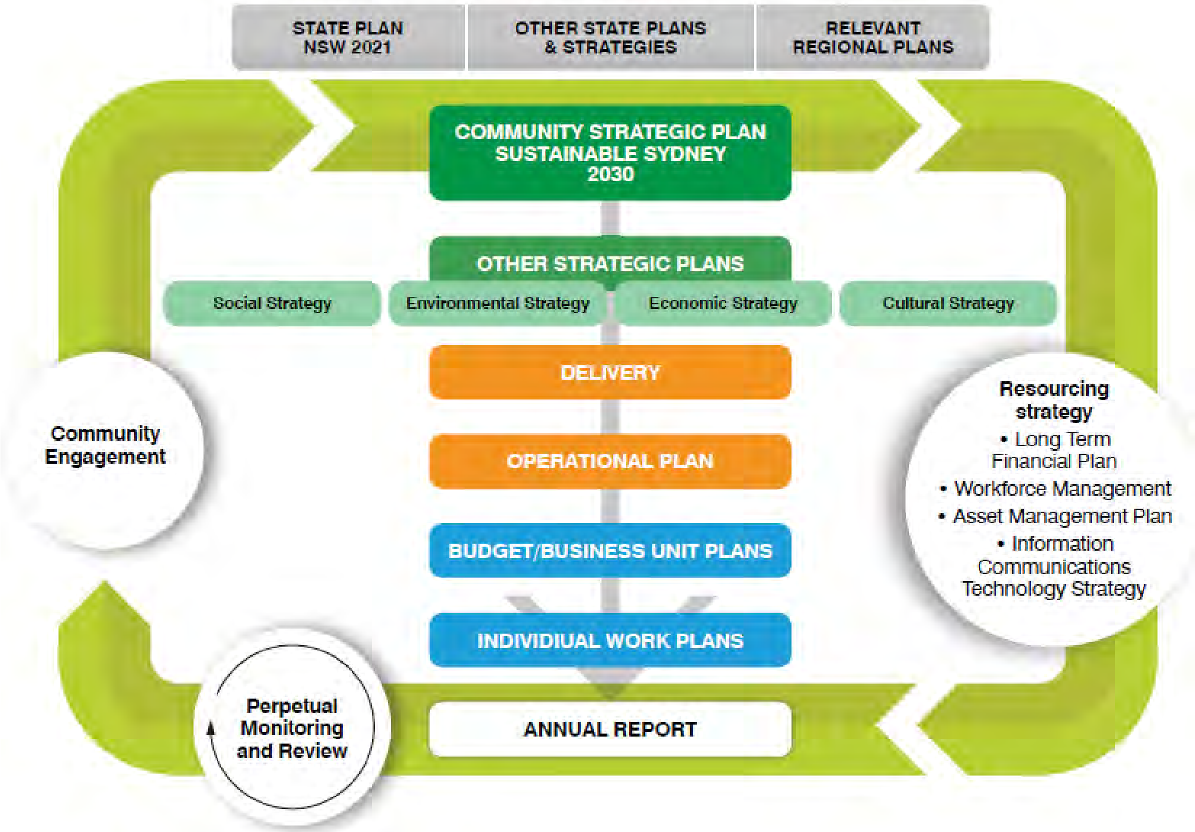

Figure 8: Integrated Planning and Reporting framework.

Figure 9: Resourcing Strategy framework.

The need to consolidate asset related information into a corporate single register and to integrate and manage that corporate information was clearly identified as a priority in the City's asset management policy and strategy. In response, the City issued a tender and subsequently purchased and progressively implemented the enterprise Confirm Corporate Asset Management System (CAMS) under contract with Pitney Bowes Software (2017).

CAMS is now an integral part of the City's suite of corporate applications and represents a long-term and ongoing commitment (over the next 10-20 years) of time, money and resources to improve knowledge and capability, develop analysis tools and meet our Integrated Planning and Reporting requirements. Figure 10 shows a schematic of the modules of CAMS. The system contains some 270,000 assets and is supported by a mobile application called ConfirmConnect.

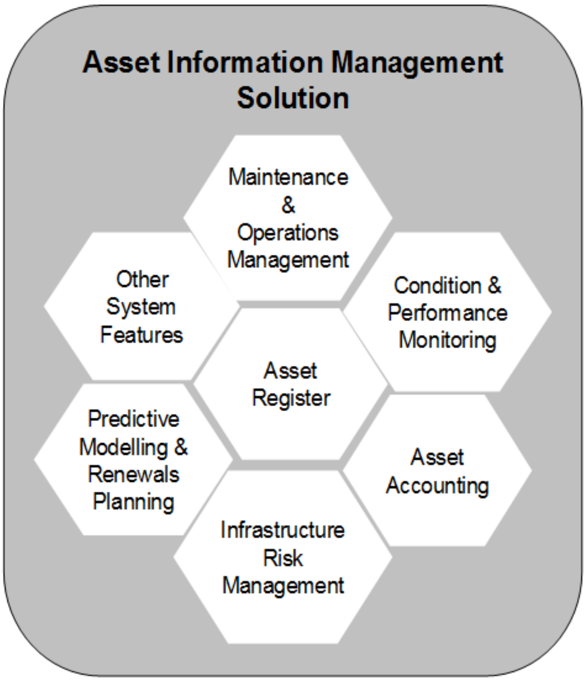

Figure 10: CAMS modules.

6 MANAGING PERMANENT SURVEY MARKS USING ASSET MANAGEMENT PRINCIPLES

In the last 2-3 years the use of CAMS has moved rapidly from a primarily desktop application to an integrated mobile solution using the ConfirmConnect application. The City has successfully implemented a full mobile solution for contractors managing street trees, which includes asset collection, inspections, maintenance management, job completion and transfer to the corporate GIS to monitor progress. It became apparent that this type of application was ideally suited to the management of permanent survey marks with the full work history and costing capability.

6.1 Asset Register

The process of developing an asset register structure was commenced by a group including the City's surveyors, asset system and GIS staff. Importantly, as not all permanent survey marks in the City are recorded in SCIMS, the register needed to be flexible enough to include

all marks that the City surveyors wanted to monitor. The register needed to include:

- Unique numbering and address information.
- SCIMS detail where appropriate.
- Specific City of Sydney data such as field books and audit areas.
- Valuation and asset condition information.

Following significant discussion, a structure was agreed and the difficult task of formatting the existing data into import files completed. The data was initially loaded into a training system so that any issues relating to the look or content could be ironed out. The task of formatting the data for import was completed by Pitney Bowes but the import itself was performed by City staff. An example of the register is shown in Figure 11.

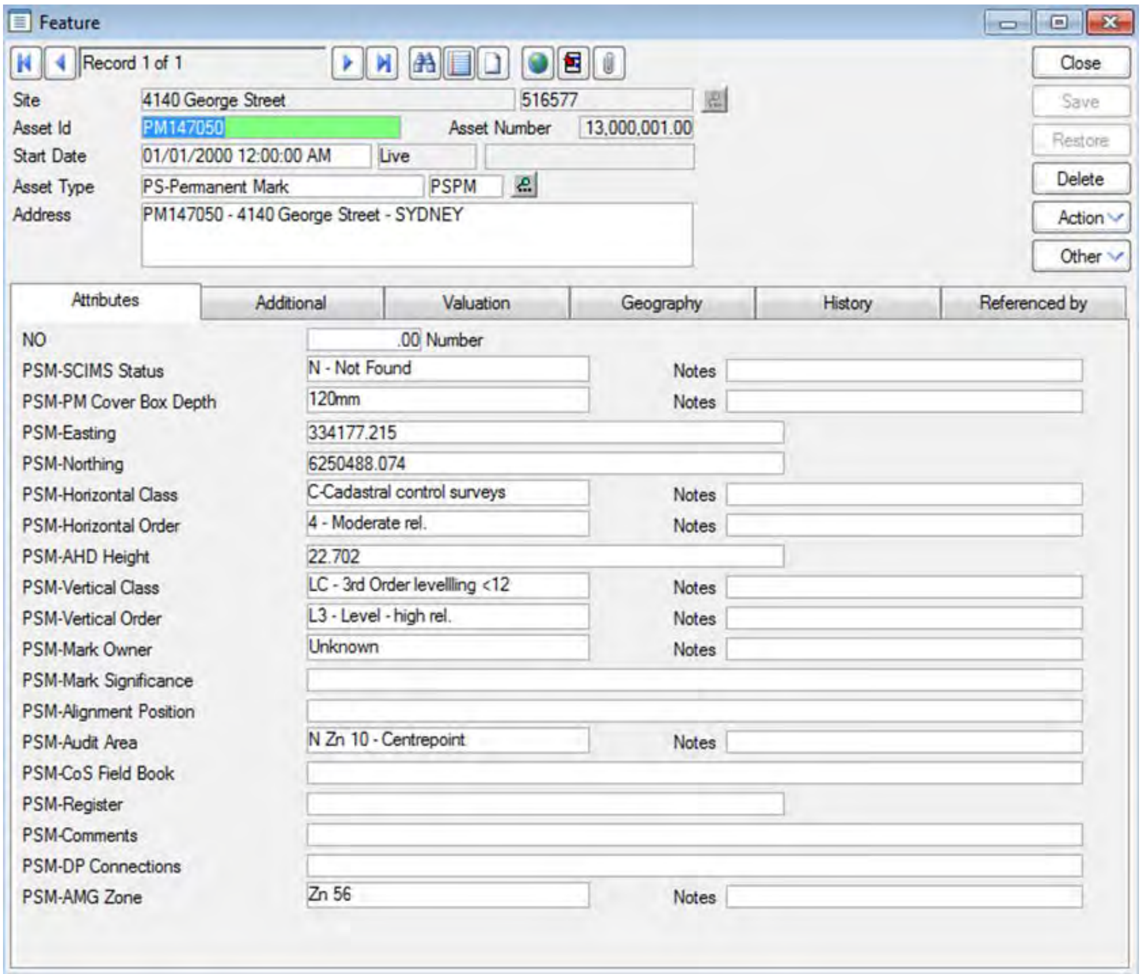

Figure 11: Survey mark asset register.

6.2 Condition Assessment

A key aim of the project was to develop condition assessment criteria and track the changes to those conditions over time. Again, the condition types and criteria were developed in-house and configured through Pitney Bowes. A general asset condition using standard asset management 1-5 scaling (i.e. 1 being very good, 2 good, 3 fair, 4 poor and 5 very poor) was included to allow analysis of the permanent survey mark network as a whole against other asset classes.

The City surveyors developed five specific condition types that represented the most important aspects of the permanent survey mark. These were box, foundation, number plate, pin and sketch plan observations, using a pass/fail basis and not any number scale. An example of the condition assessment configuration is shown in Figure 12.

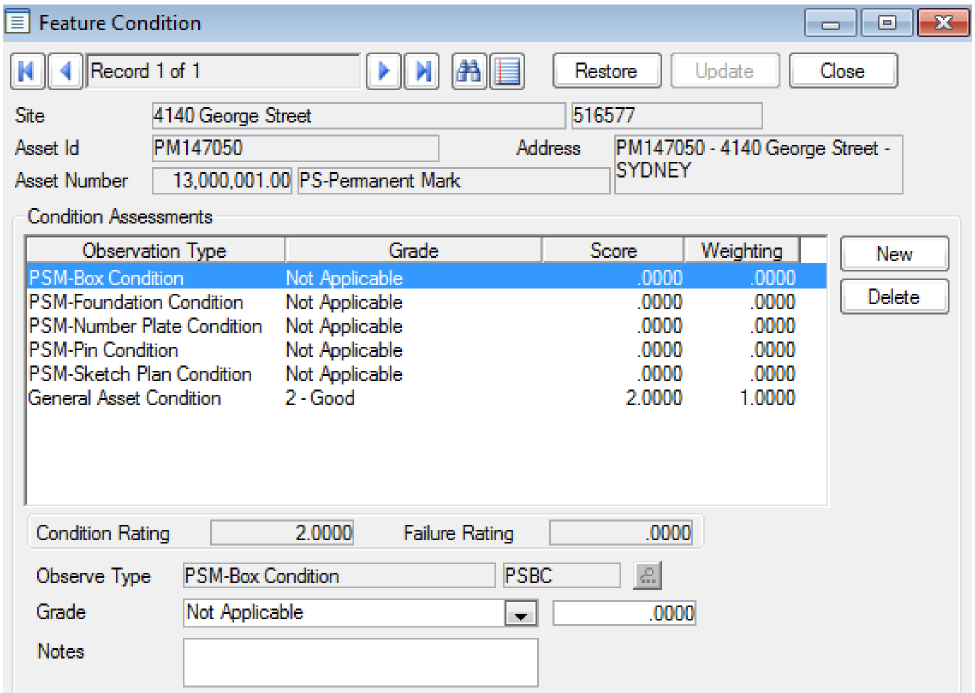

Figure 12: Survey mark asset register – condition assessment.

6.3 Defects and Schedule of Rate Items

Permanent survey mark defects are a list of things that can be damaged or missing from a mark. A Schedule of Rate (SOR) item includes the remedies for those defects and generally has a cost associated with them. Tables 1 and 2 show the configured defects and SOR items relating to permanent survey marks, respectively.

Table 1: Defect types.

defect_type_code	defect_type_name
PM01	PSM-Missing/damaged numberplat
PM02	PSM-Illegible number plate
PM03	PSM-Missing/damaged pin
PM04	PSM/SSM-Disturbed/pos.uncertai
PM05	PSM/SSM-Sketch plan outdated
PM06	PSM/SSM-Plot error
PM07	PSM BOX-Broken lid; seized
PM08	PSM BOX-Missing lid;out of pos
PM09	SSM-Illegible number
PM10	SSM-Damaged nipple/centre pt.
PM11	SSM-Missing mark (alignment)
PM12	SSM-Missing mark (other)
PM13	SSM-Damage mark seat/bedding
PM14	TS-Any defect

Table 2: Schedule of Rate (SOR) items.

sor_item_code	sor_item_name	sor_item_full_name	Unit	Quantity	Price
PSM001	PSM-Stamp new plate; install on site.	PSM - Stamp new plate; install on site.	HOUR	2	180
PSM002	PSM-Remove plate; re-stamp or stamp new plate; reinstal	PSM - Remove old plate; re-stamp or stamp new plate; reinstall on site.	HOUR	2	180
PSM003	PSM-Survey to reposition; replacement pin; notify LPI	PSM - Survey to reposition; place recovery marks; drill and set replacement pin; calculations; prepare new sketch plan; forward original plan to LPI	HOUR	10	250
PSM004	PSM or SSM-Survey to confirm coordinates	PSM or SSM - Attend site and undertake Survey to confirm position/coordinates	HOUR	4	250
PSM005	PSM or SSM-Update sketch; notify LPI	PSM or SSM - Take sufficient measurements on site; draft updated sketch plan; forward original plan to LPI	HOUR	4	250
PSM006	PSM or SSM-Estimate correct coordinates; notify LPI	PSM or SSM - Estimate correct coordinates from GIS digitisation or surveyed measurements; notify LPI of updated coordinates to appropriate level of accuracy	HOUR	1	200
PSM007	PSM BOX-Remove box; replace; make good; reinstall numbe	PSM BOX - Remove box & replace with new; make good pin, footing & surrounding surface; reinstall or replace number plate	EACH	1	2500
PSM008	PSM BOX-Survey to reposition; make good; reinstall numb	PSM BOX - Survey to reposition; place recovery marks; saw cut surround and hand excavate; inspect for pin; place box in correct location; make good pin, footing & surrounding surface; reinstall or replace pin and/or number plate	EACH	1	2500
PSM009	SSM-Grind down and re-stamp number in situ	SSM - Grind down and re-stamp number in situ	HOUR	2	180
PSM010	SSM-Re-drill in situ	SSM - Re-drill in situ	HOUR	2	180
PSM011	SSM-Survey to reposition; set new mark; notify LPI	SSM - Survey to reposition; drill & set new mark; calculations; prepare new sketch plan; forward original plan to LPI	HOUR	10	250
PSM012	SSM-Place new mark; survey; notify LPI	SSM - Place new mark in suitable location; survey to locate/coordinate; calculations; prepare new sketch; notify LPI	HOUR	5	250
PSM013	SSM-If mark stable, repair; if unstable remove and noti	SSM - If mark is stable , clean seating; inject cement into gaps; make good surrounds. If mark is unstable, remove and notify LPI of mark removal.	HOUR	2	180
PSM014	TS-Refer to NSW LPI	TS - Refer to NSW LPI	EACH	1	1

6.4 Inspection Routes

The main priority of the project is to update assets, complete inspections and register defects in the field using the mobile application ConfirmConnect. This is achieved by placing all assets on an appropriate inspection route and transferring that route to the mobile device. The inspection routes are developed from the Audit Area attribute as shown in Figure 11.

6.5 Jobs

The last part of the process is to create and complete a job to rectify any permanent survey mark defect by application of the appropriate SOR item(s). Job creation can be either through a desktop or mobile process with the preference being mobile. An example of a job screen is shown in Figure 13.

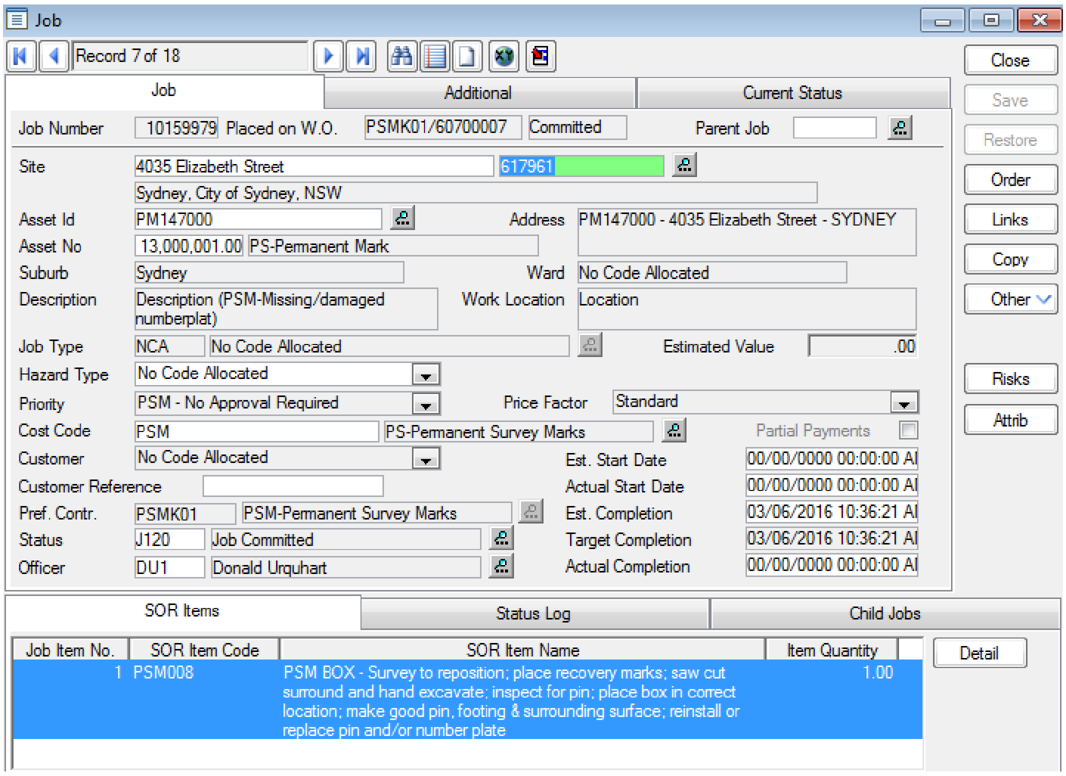

Figure 13: Job screen for a Permanent Mark.

6.6 Completing Inspection and Work using ConfirmConnect

The City surveyors can either send a complete audit area or a number of identified permanent survey marks directly to the mobile device via the desktop. The link is in real time through the 4G network. Figures 14-18 provide a number of screenshots of the mobile device screens to give the reader an idea of how the asset, condition, defect and job functionality is shown.

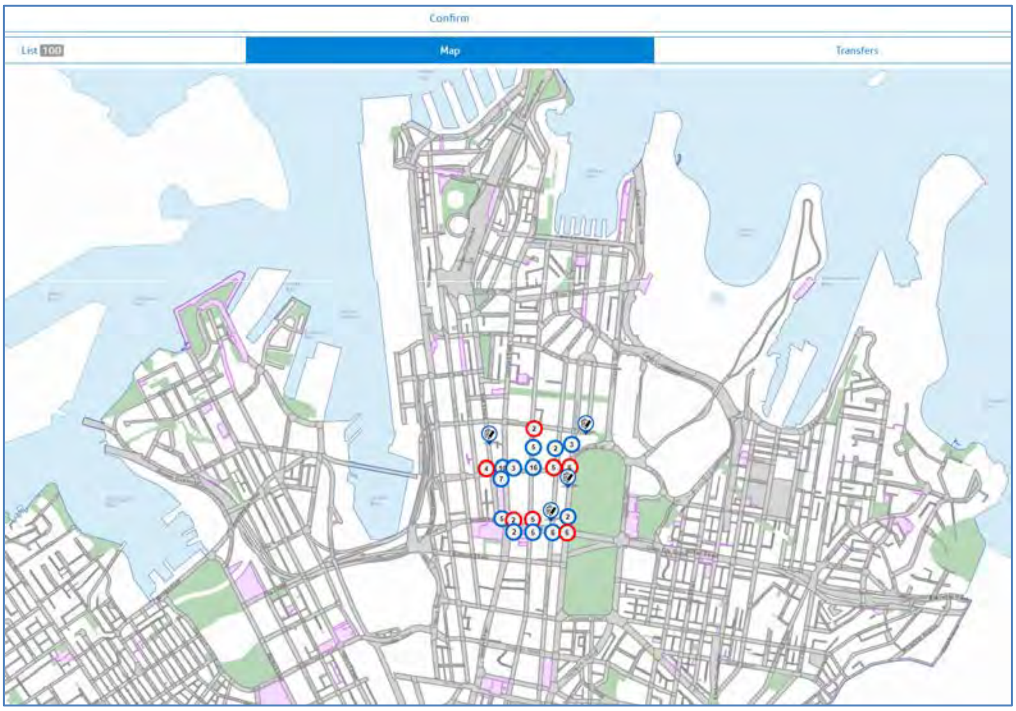

Figure 14: ConfirmConnect map.

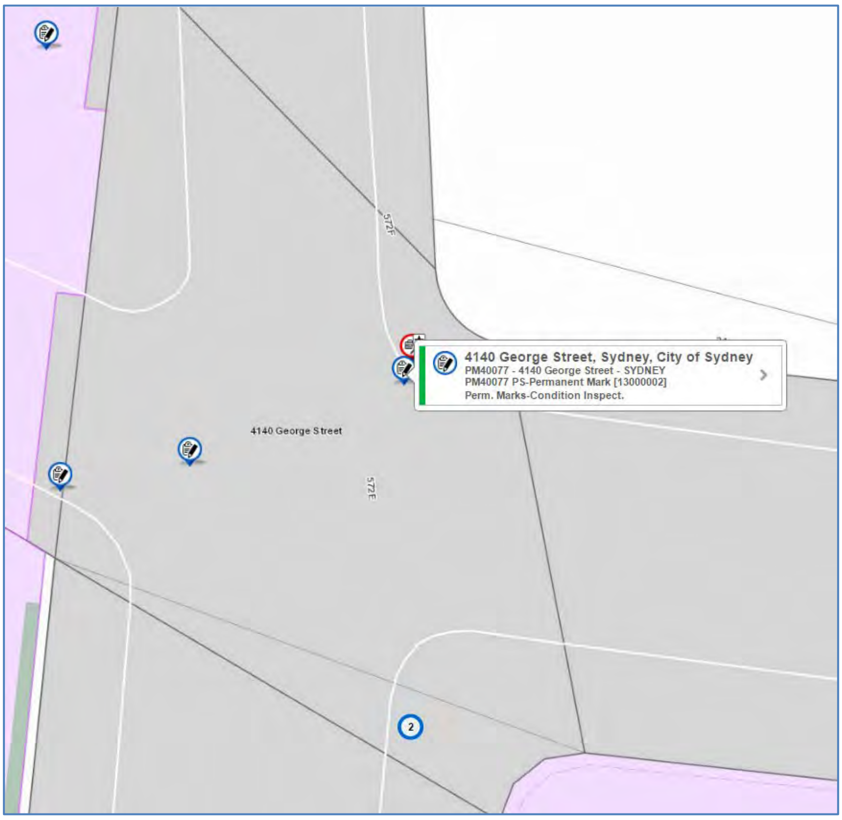

Figure 15: PM identified for inspection.

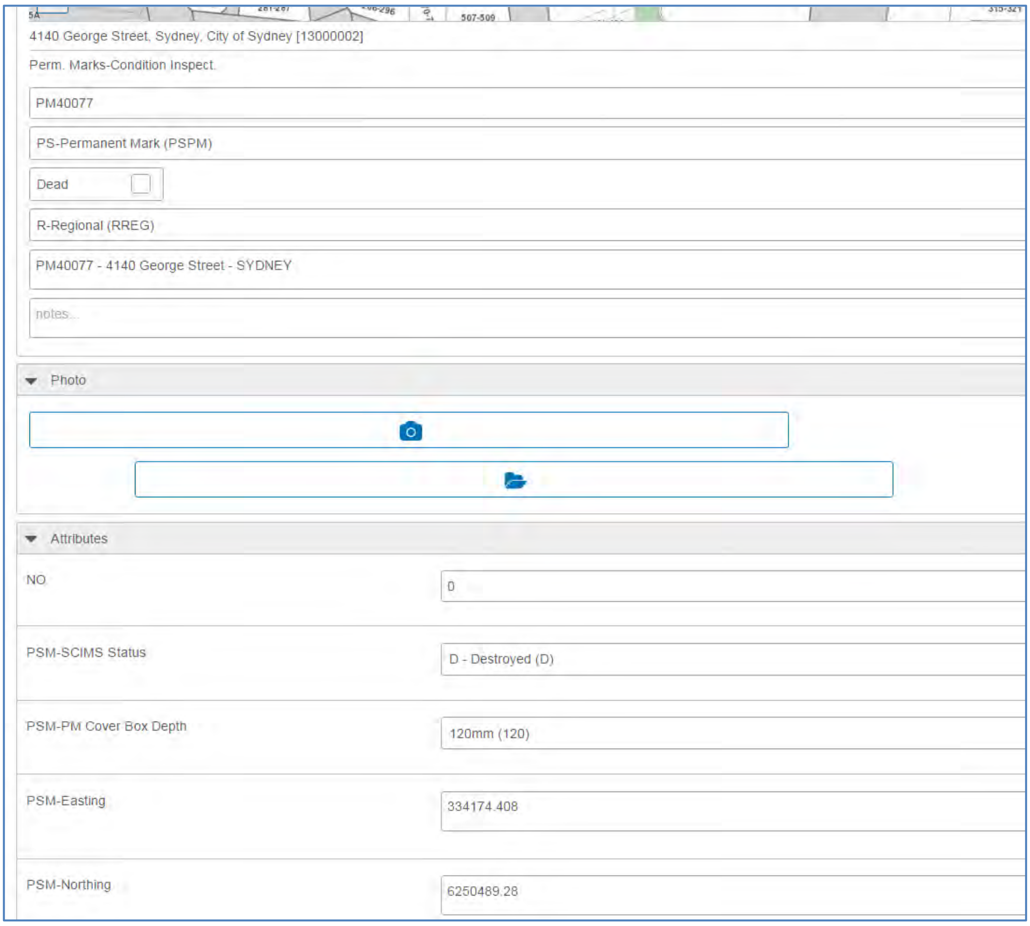

Figure 16: PM attributes – updateable in the field.

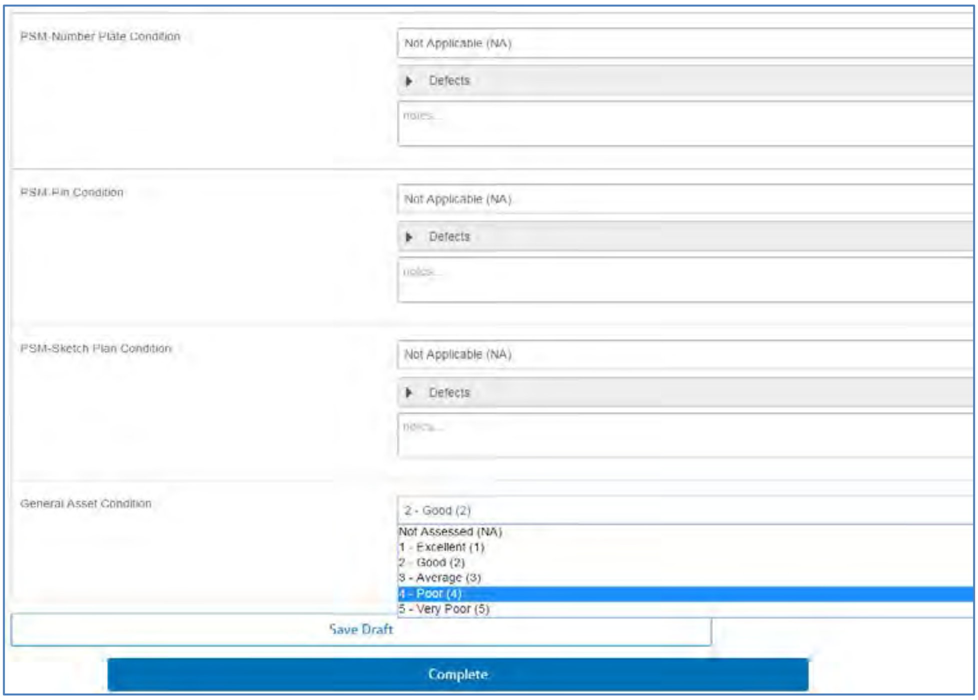

Figure 17: Condition assessment and defect logging.

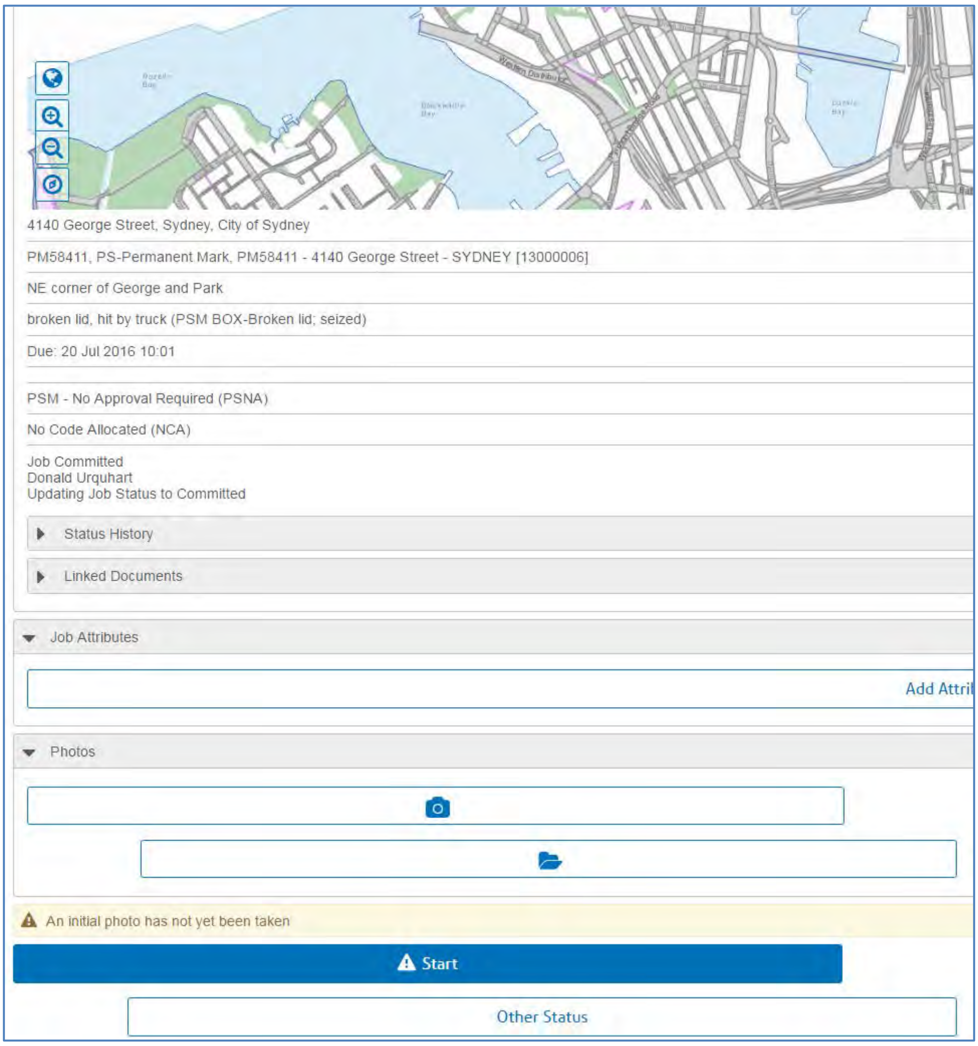

Figure 18: Job completion screen.

6.7 Result and GIS Viewing

The desired outcome of completing inspections and work activities using easier mobile applications is achieved. All condition observations are now held in individual date-stamped logs, and defect and job activity (including photos) are captured directly against the permanent survey mark.

The final part of the project was to display up-to-date permanent survey mark data on the City's internal GIS viewer. This involved not only detailed scoping of the requirements, including search capability, but specialist SQL programming skills. Fortunately, GIS and Asset staff possessed these skills, which eliminated the need to rely on external consultants.

Again after significant testing and changing, the process and data required was agreed and the interface developed. This is an overnight update process, so in essence any inspection or job activity completed yesterday is displayed in the GIS today (Figure 19).

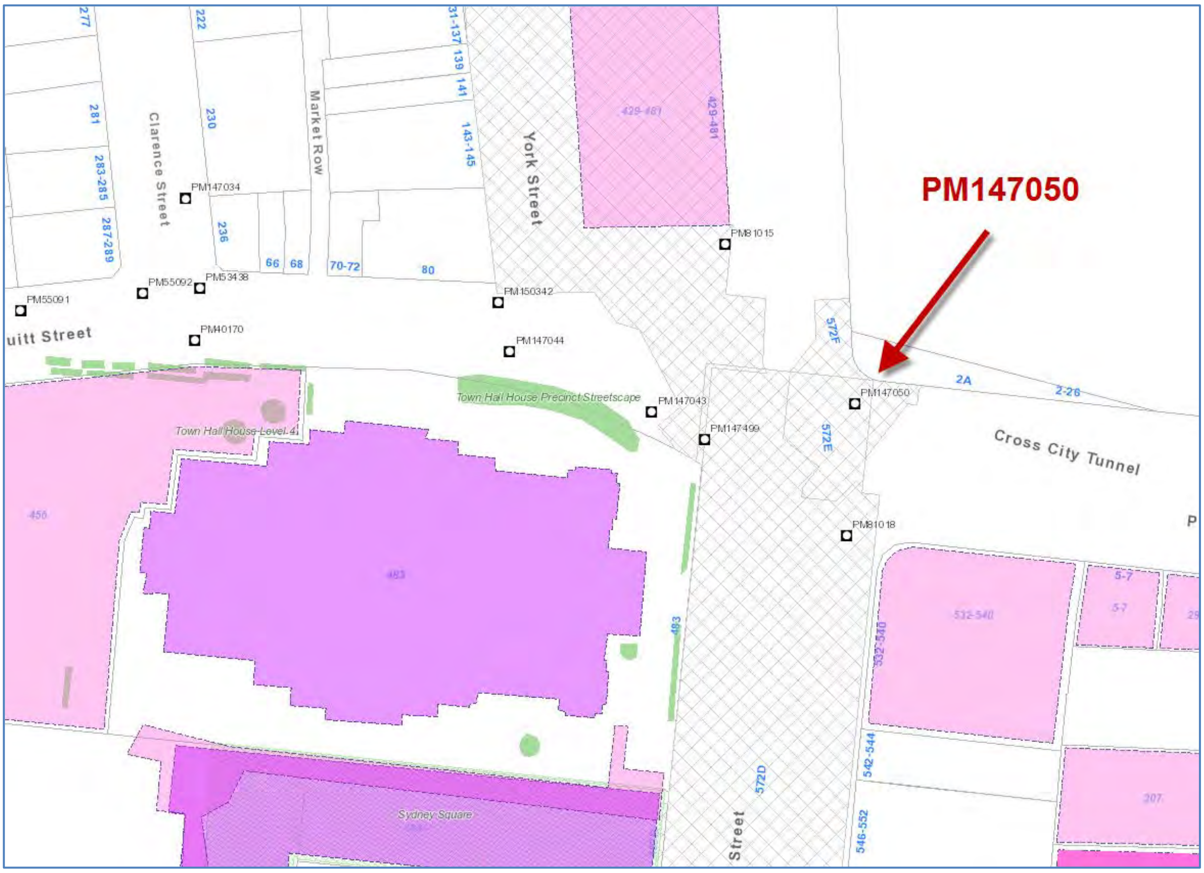

Figure 19: PM147050 and other marks shown on the GIS viewer.

6.8 Next Steps

The short-term plan for the future includes the consolidation of the use of the mobile application and incorporation of any improvements. The analysis of the captured data will guide the development, but it will take some time to understand the impact. When embedded, the distribution of permanent survey mark data to other entities will be investigated in accordance with the City's Digital Strategy.

7 CONCLUDING REMARKS

The City of Sydney has been responsible for the determination and reproduction of the alignment of the City's streets since 1879, a responsibility that is taken very seriously by the City's surveyors. The importance of the City of Sydney's permanent survey mark network to the survey infrastructure fabric of NSW is well known by surveyors at large and DFSI Spatial Services. Unfortunately, the general public and some sectors of industry do not share this understanding, and the physical existence has been threatened on a number of occasions.

This paper has detailed the history and challenges of maintaining the control network, both from a physical and organisational perspective, and the fantastic work done to replace many permanent survey marks and coordinate the network. To ensure the ongoing protection, management and improvement of the network in the future, the City has incorporated the use of modern technology and standard asset management practices to embed the permanent survey mark network into the mainstream of Council activity. The history of the permanent survey network is rich, and recognition and management of these assets using the City's normal infrastructure processes is a significant step in the continued protection and improvement of the control network.

ACKNOWLEDGEMENTS

The preservation and enhancement of the permanent survey mark network at the City of Sydney would not have been possible without the continued support of DFSI Spatial Services. The development of the mobile solution was a joint effort involving:

- City of Sydney Technical Services in City Operations.
- City of Sydney Asset Strategy and Systems in Chief Operations Office.
- City of Sydney Spatial Systems in Information Services.
- Pitney Bowes Software business support.

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Preservation of Survey Infrastructure at RMS

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ABSTRACT

This presentation provides an update of the actions and system developments undertaken by the Roads and Maritime Services (RMS) Surveying Section to ensure compliance with the preservation of survey infrastructure. It includes information about the formalisation of a Working Group with RMS Surveying and the Department of Finance, Services and Innovation (DFSI) Spatial Services representatives, the review status of Surveyor General's Directions No. 11 (Preservation of Survey Infrastructure) and 12 (Control Surveys and SCIMS) as well as RMS specifications G73 Detail Surveys and G71 Construction Surveys, new processes for large infrastructure projects, and a suite of resources available for contractors, surveyors, road designers and project managers. Several projects and lessons learnt along the way are discussed. Future work in systems development and awareness promotion activities to be undertaken in regards to the preservation of survey infrastructure is also outlined.

KEYWORDS: *Control, cadastral, marks, preservation, infrastructure*.

Creative Surveying Tools and Gadgets

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ABSTRACT

Surveyors have always been 'problem solvers' and when it comes to creative solutions, they are second to none. For example, do you need a tool to simultaneously measure the level, alignment and orientation of a tail-end drive pulley? There is nothing available commercially? Ask a surveyor and, hey presto, the problem is solved. In the 1957 classic by Nino Culotta, he describes Australians with the observation that "they're a weird mob". If he had got his first job in Australia with surveyors and not bricklayers, he might have said "they're a creative mob". This paper provides a not-so-scientific look at some innovative, interesting and downright strange tools and gadgets invented by surveyors to solve problems that came their way.

KEYWORDS: *Problem solving, innovation, surveying, gadgets.*

1 INTRODUCTION

In the 1957 classic by Nino Culotta, he describes Australians with the observation that "they're a weird mob" (Culotta, 1957). If he had got his first job in Australia with surveyors and not bricklayers, he might have said "they're a creative mob".

A number of years ago I transferred to Newcastle to work on the Kooragang Island Coal Loader Expansion Project. I was fortunate to work with and learn from two surveyors who had been involved in engineering and construction surveying for their whole careers. John and Stan were great surveyors and had that certain 'knack' for problem solving. Their customised hand-made tools, their creativity and their passion for their work was the inspiration behind many of the 'gadgets' described in this paper as well as a realisation that with a bit of lateral thinking we can solve almost anything that comes our way. Thanks, John and Stan.

This paper outlines a series of tools, gadgets and ideas and their application in surveying. It is not a scientific paper and not all the items are unique, but it is hoped you find them interesting and inspiring (and fun). To quote Albert Einstein: "Creativity is intelligence having fun".

Commercially there is a huge array of specialised tools available for surveyors. The ones I have had the opportunity to use are well made, precise and typically do a great job. And they often cost a fortune! The hand-made survey tools and gadgets described in this paper are not in the price league of commercially available tools, although they are often as well made and almost universally fit for purpose. The following scenarios look at the use of hand-made surveying tools and gadgets and where applicable comparisons are drawn to commercially available tools.

2 SURVEYING RAILS (PART A)

2.1 Scenario

Rails are not just for trains. In workshops and factories, cranes and other plant use rails and the alignment, spacing and level or grade of the rails is critical. As an example, Figure 1 shows the Leica GRP1000 (Leica Geosystems, 2017a) in use. It is practical and precise, but very expensive. Perhaps a less expensive and simpler solution would be a block of wood?

Figure 1: Leica GRP1000.

2.2 Solution

Not having a budget large enough to purchase a Leica GRP1000, an inexpensive but accurate solution was required. The tool involved three pieces of wood, two prisms and one hour of construction (Figure 2). After a small amount of calibration it was put to use and performed flawlessly.

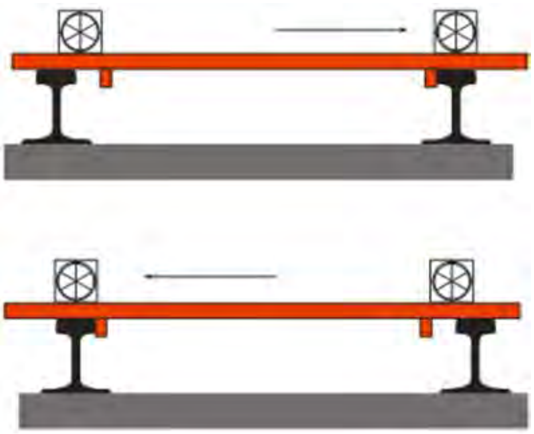

Figure 2: Rail tool.

2.3 Application

The device was placed hard up against the inside running edge of one rail and observations were taken. The rail was then pushed against the inside of the other rail and another measurement taken. Then the tool was pushed along the rail (with an attached broom handle – not shown in Figure 2) and the observations repeated.

3 SURVEYING RAILS (PART B)

3.1 Scenario

Another type of rail survey was the survey of a section of rail (about 200 m) on a mine site where we needed to find a quick and accurate way to find the centre and elevation of the rails without the hassle of physically measuring and marking up the rails every time (and all the bending over that would be involved!).

3.2 Solution

A surveyor in our team on site designed a very simple attachment for the end of a sighting pole and we had a fitter and machinist make up the attachment in a matter of an hour or so (Figures 3 & 4). The attachment was simply a small plate of steel with two vertical rods placed equidistantly about the centre and a 5/8" bolt welded in the centre (to attach it to the sighting pole). The attachment was placed on the rail and rotated until the vertical rods touched each side, automatically centring the tool.

Figure 3: Rail tool on sighting pole.

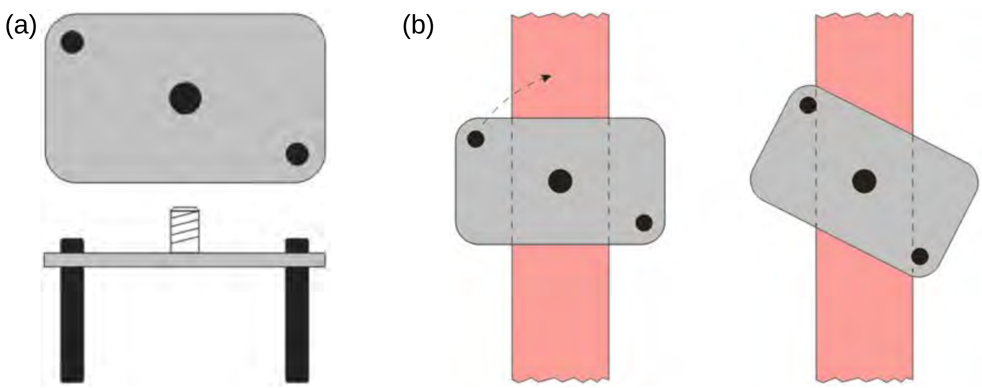

Figure 4: (a) Plan and evaluation view, and (b) rotate and self-centre.

3.3 Application

After checking the tool against observations made by measuring and marking up a section of rail and finding no errors, we proceeded to measure the 200 m of rails quickly and efficiently without any issues.

4 HIDDEN POINT (FROGS)

4.1 Scenario

Inside workshops and factories, points are often not easy to observe directly using traditional targets and sighting poles. Often the point is only accessible at an angle due to obstructions. In the scenario depicted in Figure 5, the surveyor needed to observe point A but other machinery obstructed the line of sight. Leica Geosystems (2017b) and Trimble equipment have routines that allow you to measure 'hidden points', but long before these routines were in use the 'frog' was invented. The frog was used for measuring points that were not readily accessible and was simplicity itself.

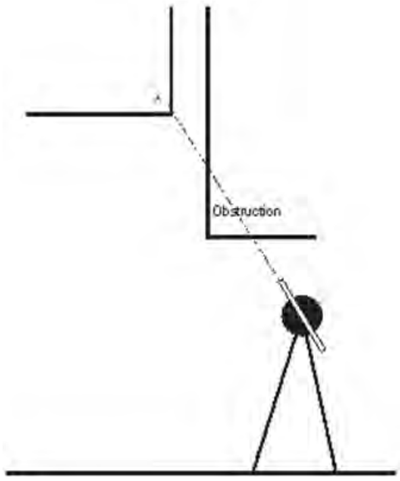

Figure 5: Obstructed observation.

4.2 Solution

The 'frog' was a rigid aluminium bar with reflective targets spaced at intervals along the face. The frog could be angled into confined spaces and held steady (typically braced) against the point to be observed. The surveyor would measure to two (preferably more) targets and record the coordinates of each target (Figure 6). The data was post-processed in a spreadsheet to calculate the hidden point's coordinates.

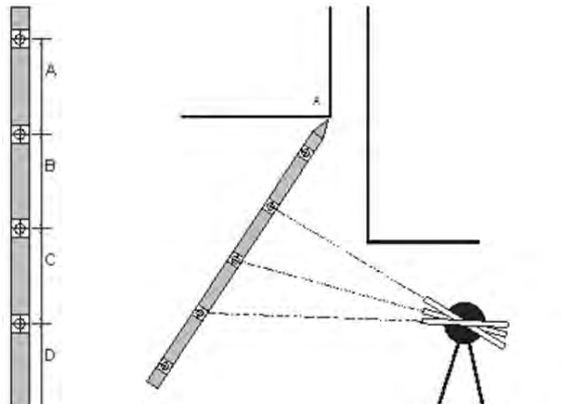

Figure 6: The 'frog' and its use.

4.3 Application

The frog was used for any situation where the point to be measured was inaccessible, but with the introduction of reflectorless Electronic Distance Measuring (EDM) technology the need for the frog became less and less.

5 HOLE CENTRES

5.1 Scenario

Often there is a need to observe the centre of bolt holes, typically at the base of a flange or tower. There are commercially available 'hemi-spheres', which are placed in the holes and give you a defined point to place a mini prism on (Figure 7). The hemi-spheres come in a number of different sizes (depending on the size of the hole) and are relatively straightforward to use, but a set can cost hundreds of dollars and they are very easy to misplace.

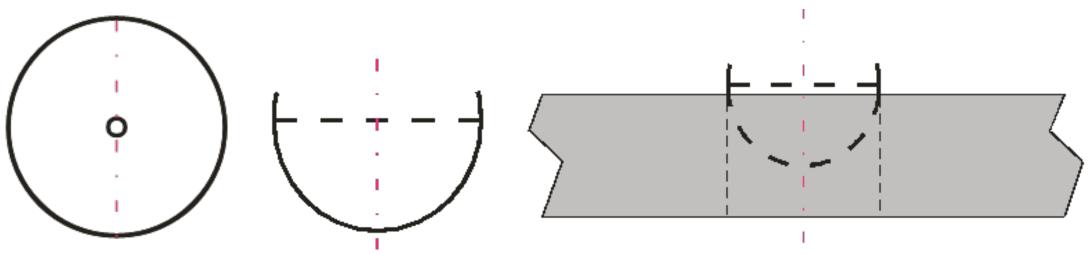

Figure 7: Plan view, side view, and hemi-sphere centred in bolt hole.

5.2 Solution

The solution was inspired by a simple kid's spinning 'top'. The 'centre top' was self-centring and by being fabricated with the angle at the point at 90º, it meant that the offset from the point of the centre top to the point of contact with the bolt hole was equal to the radius of the hole. A single observation gave the 3D coordinates of the centre of the hole at the top of the flange. The spindle of the centre top was machined to take a standard black Leica mini prism and internally threaded to also take orange Leica mini prisms (Figure 8).

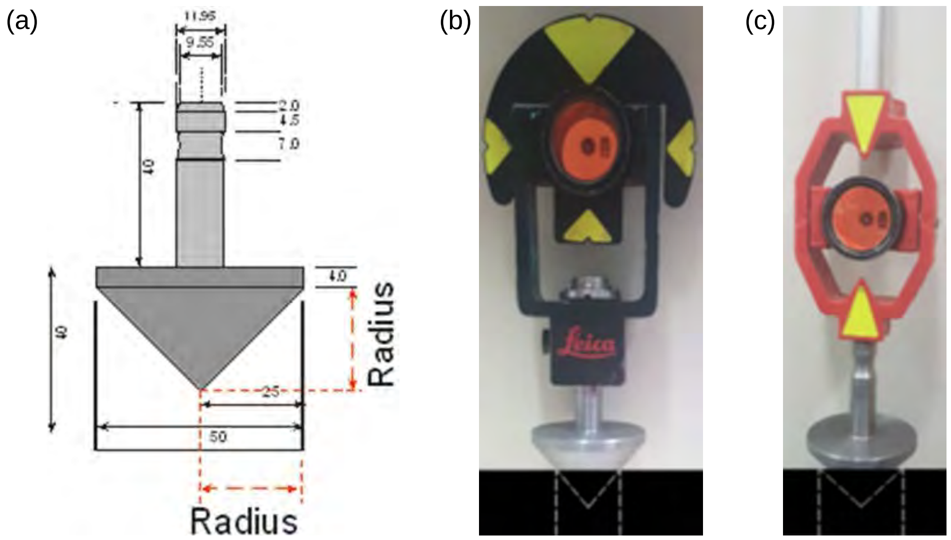

Figure 8: (a) 'Centre top' design, and holding (b) a black Leica mini prism and (c) an orange Leica mini prism.

5.3 Application

The obvious use for this gadget was for surveying bolt holes and holes in flanges. However, I have also found it a really useful way to take observations to galvanised iron pipes and as such it works brilliantly – not a bad $50 investment.

6 BOLTS

6.1 Scenario

A project that I worked on in the Pilbara involved the setting out and cast-in bolts for stackers and reclaimers on an iron ore site. There were nearly a thousand bolts to be set out in 3D (by another survey firm) prior to concreting the bolts in and then the same bolts had to be checked by us. Traditionally the centre of the bolts had to be marked so a mini prism could be used to observe the marks. This is tedious work and involves a lot of kneeling and bending (and my old bones were not really happy with this idea).

The process was facilitated to some extent by using a centre-finder to mark up the bolts (Figure 9). A centre-finder is a great tool to have and works well to find the centre of bolts or round rods. Marking up each bolt is slow, although the centre-finder makes it easier, but there had to be a better way. An alternative would be to use tools like the commercially available PQR nuts (PQR, 2017), but these still require you to screw the PQR nut onto each bolt in turn (Figure 10).

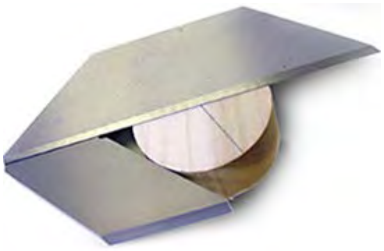

Figure 9: Centre-finder in use.

Figure 10: PQR nuts.

6.2 Solution

Given that the bolts were all the same size (that made it easier), I had fabricated an aluminium cap that fitted over the bolt and attached to a standard Leica (orange) mini prism. This 'bolt-cap' worked perfectly, so well in fact that the survey firm who was installing the bolts asked for one to be made for them too (Figure 11).

The bolt-cap was so successful that it was estimated to have saved thousands of dollars and hours of effort. There was no marking up required on the bolts and no kneeling or significant

bending involved. The bolt-cap was very stable on the bolt and a real pleasure to use. It was so efficient in terms of time, cost and ergonomics that it was nominated for a Rio Tinto productivity award.

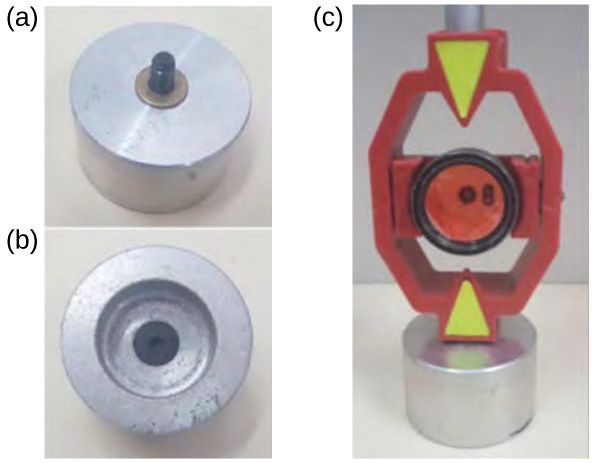

Figure 11: Bolt-cap (a) top view, (b) underside view, and (c) attached to an orange Leica mini prism.

6.3 Application

The bolt-cap was in one size and therefore only of limited use for other situations, but the concept was simple and could easily be reproduced. The time and effort saved certainly justified the $100 spent on the bolt-caps.

7 POINTERS AND REFLECTIVE TARGETS

7.1 Scenario

Often there is a need to identify a point to be surveyed. The pointer is one of the simplest yet arguably the most useful of tools I used on a number of construction sites.

7.2 Solution

The solution was to attach reflective stickers to whatever was at hand. The most useful was simply a 150 mm steel ruler with a reflective target attached (Figure 12). This can be used to point directly onto a point to be surveyed. It had the added advantage of doubling as a ruler!

Figure 12: Reflective target on ruler.

However, sometimes the mark is not always that easy to point to and I was shown a neat target made out of a piece of plastic and a reflective sticker (Figure 13). This target is placed directly over the point to be observed and read directly. The small hole in the centre allows the target to be aligned easily to a point.

Figure 13: Reflective sticker on plastic.

7.3 Application

Just about anything, anywhere, anytime.

8 PULLEY MEASUREMENTS

8.1 Scenario

While working in Newcastle, I undertook a considerable amount of engineering work involving aligning conveyor belts, drive and tail pulleys. Measuring pulleys to determine alignment, squareness and level can be a difficult task and there is not one single method for undertaking this task. What we looked for was a simple yet robust solution that would work whatever the situation.

8.2 Solution

What we came up with was a combination of Leica Reference Line and Hidden Point routines and a roller tool (Figure 14a). I have not tried the Trimble tool but I assume that it works in a similar fashion. The tool gave the position relative to the centreline of the conveyor, squareness to the centreline and level of the roller axis simultaneously.

The tool is placed against the roller hub (Figure 14b) and oriented in any angle to allow for a clear observation of the three targets at A, B and C. The targets were available to be observed straight on (right-angled targets) or the tool could be turned through 180° and observed side on (reflective targets).

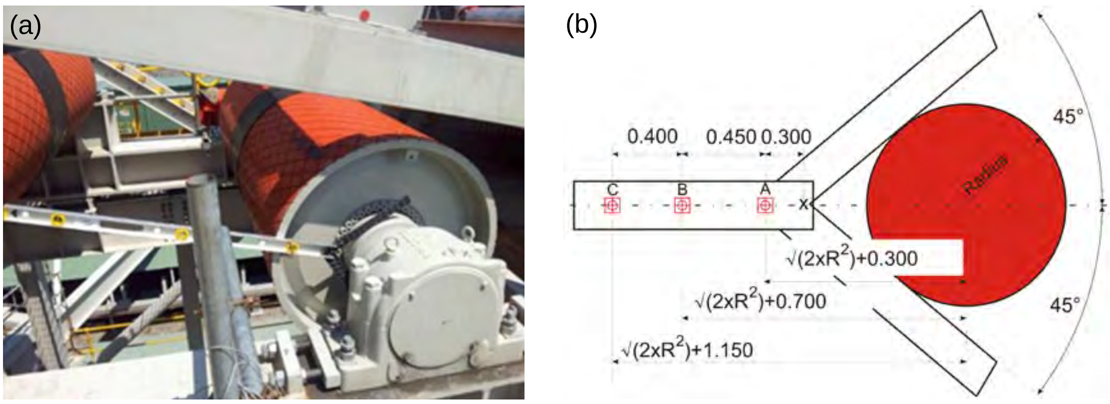

Figure 14: (a) Roller tool in use, and (b) roller tool schematic.

The procedure is as follows:

1. Measure the radius of the roller hub (actually, measuring the diameter or circumference and calculate the radius is easier). As the angle of the 'arms' is 90°, the distance from the centre of the roller is calculated to be $\sqrt{(2R^2)}$ and therefore the distances from the centre of the roller to A, B and C are calculated as shown in Figure 14b.
2. The next step is to use the Hidden Point option in the instrument to calculate the coordinates of the centre of the roller on that side of the roller. Note that observations to any two points give a solution, but observations to three points provide a check on the observations.
3. Repeat the process for the other side of the roller.

The subsequent calculation includes the following:

1. The level (or height) of the roller, i.e. difference in RLs.
2. Using the Reference Line, you can calculate the chainages either side of the roller to determine the squareness of the roller to the centreline of the conveyor.
3. The average chainage is the position of the roller and can be compared to the design position of the centre of the roller.

8.3 Application

The roller tool worked brilliantly, giving the flexibility needed in some situations where direct observations of the roller would be problematic. The advantage of the roller tool was that it could be attached to the roller and then rotated until it was in a position where all three targets were visible to the instrument. The targets did not need to be at any particular orientation (e.g. horizontal or vertical) for a solution to be determined. Given site restrictions and access issues, having a tool that could be oriented in any direction was a real bonus.

9 PIPE SURVEY

9.1 Scenario

An interesting challenge arose involving the survey of a series of 200 mm to 460 mm cast iron pipes within a factory. The location, orientation and alignment of the pipes had to be determined accurately so that a new set of pipes could be fabricated to align with the old.

9.2 Solution

The solution involved modifying a commercially available device called a 'flange wizard'. The magnetic centring heads are designed to establish and mark a centre point on a pipe or tank (Figure 15). The flange wizard was modified in order to take a black Leica prism (Figure 16), providing a simple solution that solved an interesting problem.

9.3 Application

Although designed to survey pipes, the tool can be used for a number of applications where rounded objects need to be surveyed.

Figure 15: Commercially available flange wizard (Flange Wizard, 2017).

Figure 16: Modified flange wizard.

10 MINI BIPOD

10.1 Scenario

The mini bipod is perhaps the most useful tool I use on a daily basis (by the way, I did not invent it). Often I need to observe a drill hole, nail or State Survey Mark (SSM) without setting up a full tripod and target, but want something more accurate and semi-permanent than a mini prism so you can re-observe if required.

10.2 Solution

The mini bipod shown in Figure 17 was fabricated by a local welding service and utilises a standard Leica-like round prism and mini pole (or mini pole plus extension pole). It is simple, inexpensive ($90-$120 for a set) and extremely quick to set up.

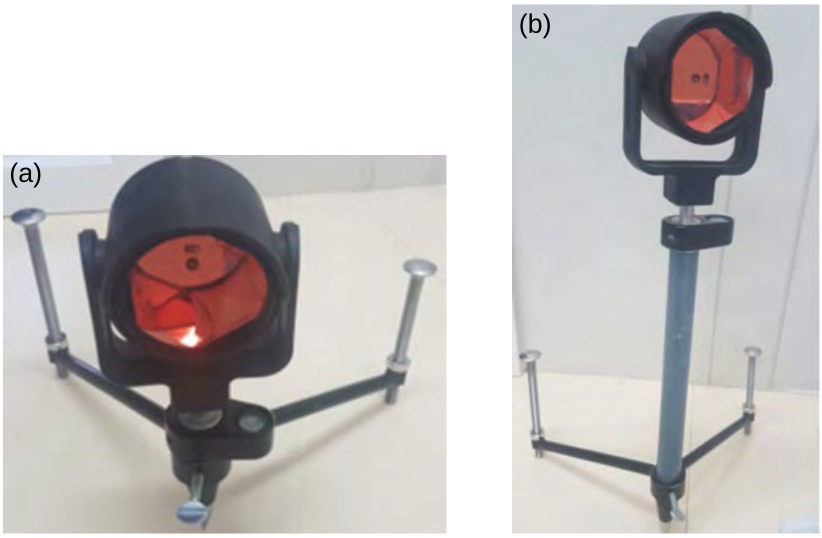

Figure 17: (a) Mini prism on mini bipod, and (b) setup including an extension pole.

10.3 Application

The mini bipod is used anywhere a quick and simple targeting system is required that gives extremely accurate results. The added advantage is that with a quick release of the mini bipod, I have a mini pole and prism at my disposal. As an interesting side note, recently I forgot to pick up a mini bipod set up on a construction site. It was sitting on a concrete slab in the middle of the site, still in position and still level three days later, a tribute to the stability of the mini bipod.

11 BLOCKS

11.1 Scenario

In engineering work, I often use simple scratch marks on concrete, or paint pen marks on columns or whatever surfaces are at hand. They are durable stations that do not obstruct anything. It is desired to have a quick and simple target that gives accurate results.

11.2 Solution

The 'blocks' as they are called are fabricated from 30 mm galvanised angle iron and 30 mm reflective targets. They could be placed on marks or stuck to columns with super-strong magnets (Figure 18). Cheap, incredibly easy to manufacture, accurate and durable, they are an essential addition to my field bag.

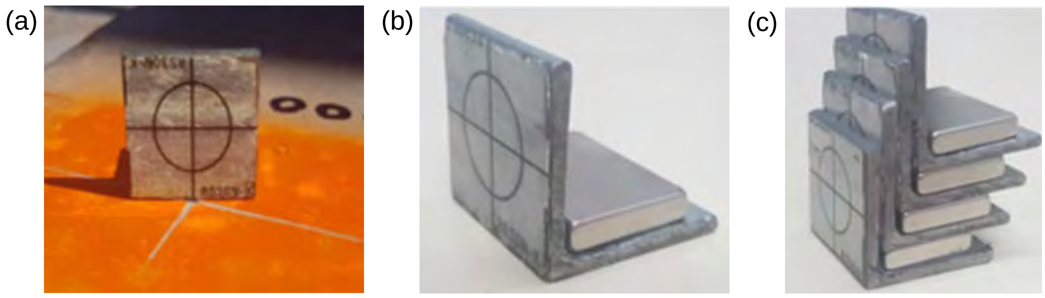

Figure 18: (a) Block set up on a scratch mark, (b) block with magnet, and (c) blocks and magnets.

11.3 Application

The blocks are simple targets for flat surfaces or steel columns. They are very quick to set up, inexpensive (less than $3 each) and very accurate. When working on construction sites, setting up a tripod and target can be an issue, so a block is a great solution.

12 CONCLUDING REMARKS

The tools described in this paper are not anything special – just simple, creative and inexpensive solutions to interesting problems. Most surveyors will have similar tools or experiences, and this paper salutes their creativity and imagination as much as John's, Stan's and mine.

Whether it is creating survey tools, solving challenging business problems or finding left-field solutions to problems in the field (often on the fly), surveyors are a creative mob and probably always have been. Who else would have thought of using the crinoline wire out their wife's skirt to make a surveyor's measuring wire (Culture Victoria, 2017)? Nino Culotta would have been proud, probably Albert Einstein too (Figure 19).

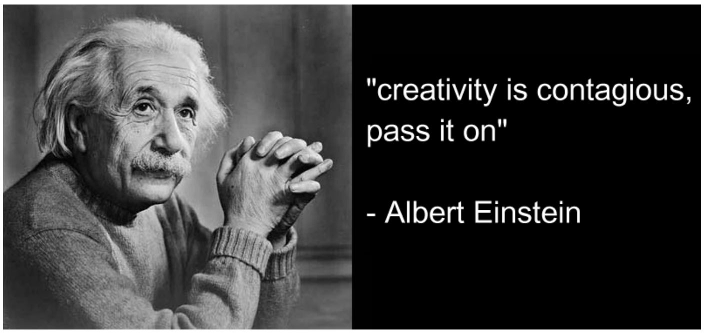

Figure 19: One of Albert Einstein's quotes on creativity.

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