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16-18 MARCH 2015

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Editorial

These proceedings contain the papers presented at the Association of Public Authority Surveyors Conference (APAS2015), held in Coffs Harbour, NSW, Australia, on 16-18 March 2015. Papers were not peer-reviewed but have been subject to changes made by the Editor.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:

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APAS is not responsible for any statements made or opinions expressed in the papers included in these conference proceedings.

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Monitoring Surveys on the Illawarra Railway

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ABSTRACT

The Illawarra Railway, south of Waterfall and as far south as Austinmer, was constructed in country that is generally rugged and unstable. The railway winds its way through the Royal National Park through some deep cuttings, high embankments and tunnels before emerging at Stanwell Park where it hugs the geologically unstable coastal area beneath the Illawarra escarpment. There has been a long history of movement of the infrastructure which was exacerbated by the engineering works associated with the electrification of the railway in the early 1980s. Since that time, multiple sites have required survey monitoring. Most have had extensive and expensive remedial works carried out, but to this day there are still many sites that are regularly monitored to detect possible embankment movement and to ensure the risk is properly managed. This presentation will outline the types of locations, the monitoring techniques involved and how the techniques, equipment used and documentation have evolved over the last 30 years.

KEYWORDS: *Illawarra, monitoring, history, current.*

Best Practice: Performing EDM Calibrations in NSW

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ABSTRACT

The Surveyor General of New South Wales is a verifying authority for reference standards of length measurements under the National Measurement Act 1960 and responsible for ensuring that surveyors use verified measuring equipment. To this end, the Surveying and Spatial Information Regulation 2012 requires surveyors to verify their Electronic Distance Measurement (EDM) equipment in relation to an Australian standard of measurement of length at least once a year. In order to assist the profession in meeting this requirement, Land and Property Information (LPI) provides and maintains several EDM baselines across the state. LPI is currently in the process of improving this infrastructure by upgrading existing baselines and building new baselines for the calibration of EDM instruments. This paper briefly presents the current status of EDM baseline infrastructure in NSW and outlines best practice guidelines for EDM calibrations in NSW. These proposed guidelines are expected to flow on into the next update of Surveyor General's Direction No. 5 (Verification of Distance Measuring Equipment).

KEYWORDS: *EDM, calibration, best practice, metrology, NSW.*

1 INTRODUCTION

Legal metrology covers all measurements carried out for any legal purpose, including measurements that are subject to regulation by law or government decree. The National Measurement Act 1960 provides the legal basis for a national system of units and standards of measurement of physical quantities (Australian Government, 2013). This Act is administered by the National Measurement Institute (NMI), which may in turn appoint organisations as verifying authorities under the provisions of Regulation 73 of the National Measurement Regulations 1999 (Australian Government, 2014). As such, the office of the Surveyor General of New South Wales (NSW) has been appointed as a verifying authority for length measurement standards and is formally accredited by the National Association of Testing Authorities (NATA) for its technical competence in providing calibration services in accordance with the requirements of AS ISO/IEC 17025 *General Requirements for the Competence of Testing and Calibration Laboratories*.

Practising surveyors in NSW are subject to the Surveying and Spatial Information Act 2002 (NSW Legislation, 2014a) and the Surveying and Spatial Information Regulation 2012 (NSW Legislation, 2014b). The latter states, among other things, that a surveyor must not use any Electronic Distance Measurement (EDM) equipment unless it is verified against the state primary standard of measurement of length by using pillared baselines, at least once every year and immediately after any service or repair. Verification is also strongly advised after the instrument receives rough treatment or hard knocks, or before it is being used for high-

accuracy surveys. This instrument verification establishes traceability of its measurements to the national standard, and consequently strengthens the validity of these measurements if questioned in a court of law.

In order to assist the surveying profession in meeting the legal requirements, the Surveyor General has established and continues to maintain several EDM baselines throughout the state. The field procedures prescribed for EDM calibrations in NSW are documented in Surveyor General's Direction No. 5: Verification of Distance Measuring Equipment (LPI, 2009). This paper provides updated best practice guidelines for EDM calibrations in NSW, which are expected to flow on into the next update of Surveyor General's Direction No. 5. It should be noted that these guidelines do not include reflectorless EDM observations.

2 EDM INSTRUMENT ERRORS AND CORRECTIONS

The calibration of an EDM instrument is performed in order to determine the instrument errors, which can be used to monitor the performance and reliability of the EDM instrument over time and assess its precision against the manufacturer's specifications. If significant, these instrument errors should be accounted for by applying corrections to measurements taken subsequent to the calibration. If the calibration is performed on a verified baseline (i.e. a baseline with a current Regulation 13 certificate) to a prescribed level of precision, the EDM instrument is considered to be standardised. The three distinct systematic errors that may occur in EDM instruments are the zero error (also known as index error), the scale error, and the cyclic error (also known as short periodic error) (Rüeger, 1996).

2.1 Additive Constant (Correction to Zero Error or Index Error)

All distances measured by a particular instrument/reflector pair are subject to a constant error, which is caused by three factors:

- Electrical delays, geometric detours and eccentricities in the EDM instrument.
- Differences between the electronic and mechanical centres of the EDM instrument.
- Differences between the optical and mechanical centres of the reflector.

In other words, the error is mainly caused by the distance measuring reference points in the EDM instrument and at the reflector not being coincident with the vertical axes at either end of the measured line. This error may vary with a change of reflector, after receiving jolts, with different instrument mountings and after service. The additive constant or zero/index correction is an algebraic constant (often stated in mm) to be applied directly to every measured distance.

2.2 Scale Error

The scale error is linearly proportional to the length of the measured line and is caused by:

- Variations in the modulation frequency of the EDM instrument.
- Incorrect modelling of the atmosphere, i.e. errors in the measured temperature, atmospheric pressure and humidity, which affect the velocity of the signal propagation.
- Non-homogeneous emission/reception patterns from the emitting and receiving diodes (phase inhomogeneities).

The scale factor (or scale correction) is generally expressed in parts per million (ppm) and also applied directly to every measured distance.

2.3 Cyclic Error (or Short Periodic Error)

The cyclic error is caused by electrical and optical interference within the EDM instrument. It varies across the modulated wavelength and is usually sinusoidal in nature with a wavelength equal to the unit length of the EDM instrument. The unit length is the scale on which the EDM instrument measures the distance and is derived from the fine measuring frequency. The unit length is equal to exactly one half of the EDM instrument's modulation wavelength (Rüeger, 1996).

As the cyclic error repeats itself for every unit length contained within a measured distance, its sign and magnitude vary depending on the length measured. The magnitude of the error could be in the order of 5 mm to 10 mm. However, in modern EDM instruments it is usually less than 2 mm and therefore negligible. It should be noted that the cyclic error can increase in magnitude as the instrument's components age.

3 CURRENT AND FUTURE EDM BASELINE INFRASTRUCTURE

The Surveyor General has established several EDM baselines consisting of between four and seven concrete pillars throughout NSW. Current best practice has established that EDM baselines should consist of at least five (and preferably six or seven) pillars to increase the number of distances observed, thereby allowing a more reliable determination of the instrument correction due to higher redundancy. As a result, and on behalf of the Surveyor General, Land and Property Information (LPI) is currently in the process of rationalising and improving its EDM baseline infrastructure by upgrading existing baselines to include more pillars and/or building new 7-pillar baselines. A detailed description of the substantial issues that need to be considered in the design and construction of a state-of-the-art EDM baseline is given in Ellis et al. (2013). Depending on the location of the baseline, particular environmental aspects may also have to be considered (Janssen, 2013). It should be noted that the use of longer EDM baselines allows more distances to be observed, thus increasing redundancy and providing considerably more reliable EDM calibration results.

Figure 1 illustrates the location of the 15 EDM baselines presently maintained in NSW. The latest addition is the new 7-pillar Seaham baseline (released in December 2014), replacing the 4-pillar baseline at Newcastle. Efforts to upgrade the 4-pillar Armidale baseline to include seven pillars and establish a new 7-pillar baseline at Coffs Harbour (replacing the 4-pillar baseline at Grafton) are well underway. In addition, it is planned to upgrade the 4-pillar Wollongong baseline to 7 pillars and establish at least two 7-pillar baselines on the South Coast.

All EDM baselines in NSW (current and those under construction) follow the Heerbrugg design (also known as Schwendener design), which features an almost equal distribution of the distances measured in all combinations over the baseline length as well as over the unit length of the EDM instrument and permits the detection of all distance-dependent errors, including cyclic errors (e.g. Schwendener, 1972; Rüeger, 1996).

Figure 1: Location of current EDM baselines in New South Wales.

LPI verifies these baselines on a 2-yearly basis with precise EDM instrumentation carrying a current Regulation 13 certificate issued by the National Measurement Institute (NMI). It should be noted that the associated meteorological equipment is also calibrated against industry standards. This process determines the 'true' inter-pillar distances and establishes traceability because the EDM baseline becomes a subsidiary standard of the International Metre. In accordance with the appointment as a verifying authority for length measurement standards, the least uncertainty quoted for the verified inter-pillar distances is currently 0.5 mm + 1.3 ppm at the 95% confidence interval. The current measurement report for each baseline can be found on the LPI website (LPI, 2015a).

4 ACCESS TO EDM BASELINES IN NSW

Access to all EDM baselines in NSW is restricted to authorised personnel, and it is mandatory to book access via the EDM Baseline Booking System. This free online booking system is available on the LPI website (LPI, 2015b) and allows registered users to reserve a particular time slot at the desired baseline in advance.

The booking process is simple and straightforward, comparable to booking a hotel room online. It consists of the following three simple steps (Figure 2):

1. Select a booking date.
2. Select an EDM baseline.
3. Select an available booking time slot.

Once the booking is finalised, an automatic confirmation will be sent by email, also outlining the general and baseline specific conditions of use that had to be accepted during the booking process. The user is required to carry a printout of this booking confirmation with them at all times when on the baseline site. This will provide proof of approved access to the baseline for the specified time period.

The EDM Baseline Booking System facilitates efficient and effective use of existing and future baseline infrastructure in NSW. By allowing LPI to monitor the frequency of use of each baseline, it also assists LPI in making more informed decisions regarding the state's EDM baseline infrastructure.

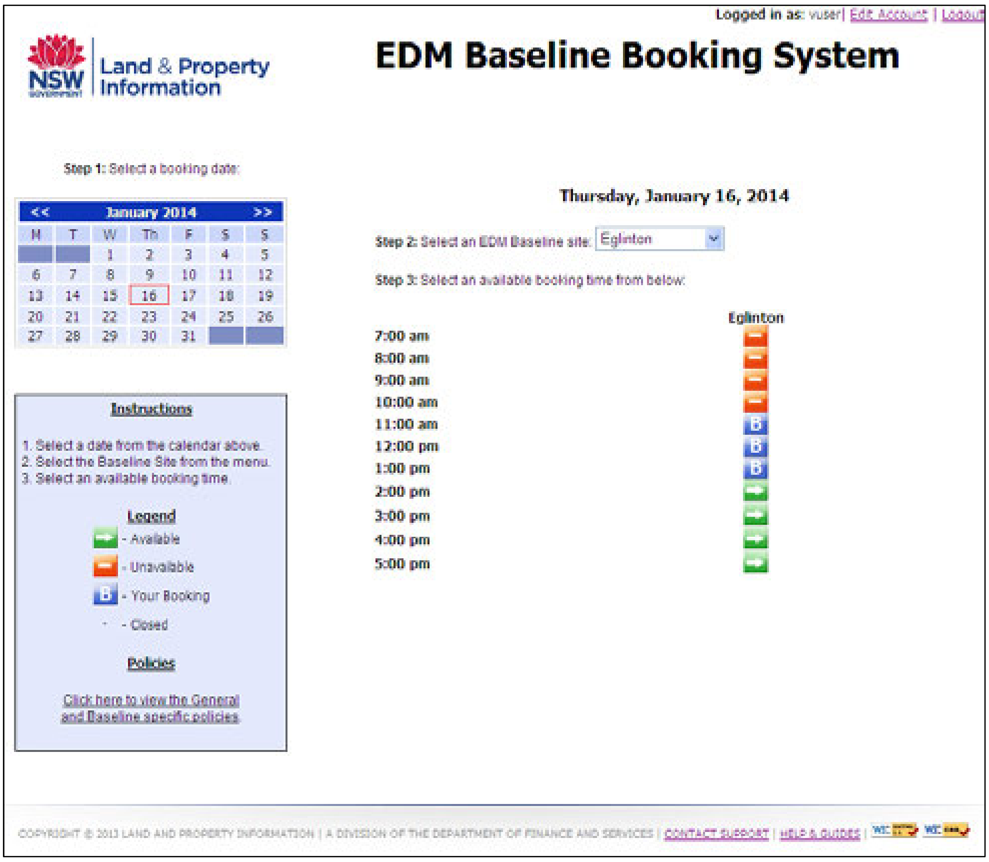

Figure 2: Main page of the online EDM Baseline Booking System (http://lpi.nsw.gov.au/edmbooking).

5 BEST PRACTICE FOR EDM CALIBRATIONS IN NSW

The calibration of an EDM instrument on a verified baseline determines the corrections that need to be applied to the instrument in order to obtain the 'true' inter-pillar distances, thereby establishing traceability of its measurements to the national standard. Most jurisdictions have produced guidelines on how to perform EDM calibrations (e.g. LPI, 2009; Queensland Government, 2012; ACT Government, 2014; Land Victoria, 2014), based on the recommendations made by Rüeger (1984). This section updates and expands the current best practice guidelines available in NSW.

It is assumed that there is no pillar movement between the time of baseline verification (performed by LPI) and the time of the EDM calibration undertaken by the user. While it is recognised that pillars do demonstrate seasonal movement in some cases, generally this

movement is too small to have any significant effect. Baseline stability is closely monitored by LPI to ensure that calibrations can be performed to the required precision. If it is suspected that pillar movement has occurred, it should be reported to LPI via EDMcal@lpi.nsw.gov.au for immediate action and resolution.

5.1 Preparation of Equipment

1. Obtain the latest EDM baseline measurement report detailing the current distances, reduced levels and access details from the LPI website (LPI, 2015a).
2. Use the EDM Baseline Booking System (LPI, 2015b) to book access to the baseline (see section 4).
3. Check the levelling bubbles on all tribrachs, reflectors and the total station, and adjust if necessary before observing the baseline. Levelling of the instrument and reflectors is critical during calibration.
4. Verify the thermometer(s) and barometer(s) against a certified standard. The collection of accurate meteorological data is essential for a reliable EDM calibration.
5. Ensure that the EDM battery is fully charged prior to carrying out the calibration.
6. All reflectors should be marked with a unique identification number. Only one of these reflectors is to be used for the EDM calibration observations.
7. It is recommended to use the EDM calibration recording sheet available from LPI (2015a) for the recording of baseline observations in the field.

5.2 Observation Procedure

1. Standard Work Health and Safety (WHS) principles must be observed. For example, this includes obeying road rules, not obstructing traffic near the baseline and wearing personal protective equipment.
2. Any general and baseline specific conditions of use must be obeyed. These conditions are detailed in the booking confirmation email, a printout of which must be carried on site.
3. Upon arrival at the baseline, each pillar must be checked for damage, disturbance or obstruction. Remove any protective pillar caps (and replace these after completion of the field work). If minor clearing of vegetation is required, do so in the appropriate manner and adhere to any restrictions and/or process that may be applicable.
4. The instrument and meteorological equipment should be shaded by an umbrella. Note that most EDM instrument specifications refer to a temperature range of -20°C to +50°C. However, the temperature inside an EDM instrument in direct sunlight on a hot summer's day can exceed this temperature range.
5. Before commencing measurement, the EDM instrument should be carefully levelled and allowed a 'warm up' period if recommended by the instrument manufacturer.
6. Set the additive constant and the atmospheric correction (ppm) to zero. Some EDM instruments only accept the input of ambient temperature and pressure readings in lieu of a ppm setting. When calibrating these instruments, the operator should refer to the instrument manual and input a temperature and pressure which corresponds to the reference refractive index for that particular instrument. This is the temperature and pressure at which the instrument applies a zero ppm correction to measured distances.
7. If possible, set the instrument to display distances to four decimal places of a metre rather than three.
8. The height of instrument and the height of reflector above the pillar plate are to be measured to an accuracy of one millimetre. These heights are combined with the height of the pillar plate to reduce distances to the horizontal.

9. All measurements should be made to one, uniquely numbered reflector. Note that a separate tribrach may be fixed to each of the pillars and the single reflector located in each tribrach in turn. Centring errors caused by the tribrach are very small in relation to the magnitude of other instrument/reflector errors and may be ignored.
10. Point the instrument and reflector as prescribed by the manufacturer in order to maximise the return signal strength.
11. The observation sequence should be chosen so that the shorter lines are measured first and last. For baselines consisting of 6 or 7 pillars, it is sufficient to observe the baseline in one direction only (generally in the forward direction). On a 7-pillar baseline, this translates into the following sequence of 21 distances (Figure 3): 1-2, 1-3, 1-4, 1-5, 1-6, 1-7; 2-7, 2-6, 2-5, 2-4, 2-3; 3-4, 3-5, 3-6, 3-7; 4-7, 4-6, 4-5; 5-6, 5-7; 6-7, where '1-2' represents the observation from pillar 1 to pillar 2, etc. For baselines consisting of 4 or 5 pillars, it is necessary to observe all inter-pillar distances in both the forward and reverse direction in order to achieve reasonable redundancy. On a 4-pillar baseline, this translates into the following sequence of 12 distances (Figure 4): 1-2, 1-3, 1-4; 2-4, 2-3, 2-1; 3-1, 3-2, 3-4; 4-1, 4-2, 4-3.
12. A minimum of five individual slope distances should be measured to the same single reflector, re-pointing after each measurement. This will allow the instrument to go through the initialisation procedure and reset the signal strength for each measurement. The instrument should not be set to display the mean of a set of five measurements in lieu of five individual readings unless this procedure is repeated five times independently. Horizontal distances are computed more accurately using the known pillar heights. Recording of the horizontal distance displayed by the instrument or reducing the slope distances to the horizontal using the zenith angle should only be used for a check on field procedure or on-board computation. The following sources of error may occur if horizontal distances are calculated from zenith angles either manually or by automatic reduction in the EDM instrument: pointing error, vertical circle index error, variation in reduction formula used in different instruments, round-off errors after automatic computation in the EDM instrument.
13. The temperature and atmospheric pressure at both the instrument and the reflector should be measured to an accuracy of at least 0.5°C and 1 millibar (mb) respectively, using calibrated thermometers and barometers. Temperatures should be measured at the height of instrument and reflector to minimise the effect of radiated heat from the ground. Note that pressure may be measured at the instrument only, provided the baseline is not located in steep terrain.
14. When transporting the instrument between pillars, ensure that it is kept shaded from direct sunlight.
15. Once all inter-pillar distances have been measured to the one uniquely numbered reflector, compare this reflector with the remaining reflectors by measuring to each in turn. This should be carried out on the shortest line and by comparing the slope distances. However, if the reflectors vary in height, measurements should be reduced to the horizontal before the comparison is made. This comparison is important when using different makes of reflector but can also be significant when different reflector holders of the same make are used, e.g. single reflector holders compared with triple reflector holders. Where found to be significant, variations should be applied as corrections to the additive constant for each reflector concerned. It is for this reason that all reflectors should be uniquely numbered. Subsequent calibrations of the EDM instrument should be performed using the same uniquely numbered reflector where possible in order to compile a calibration history for the instrument/reflector combination.

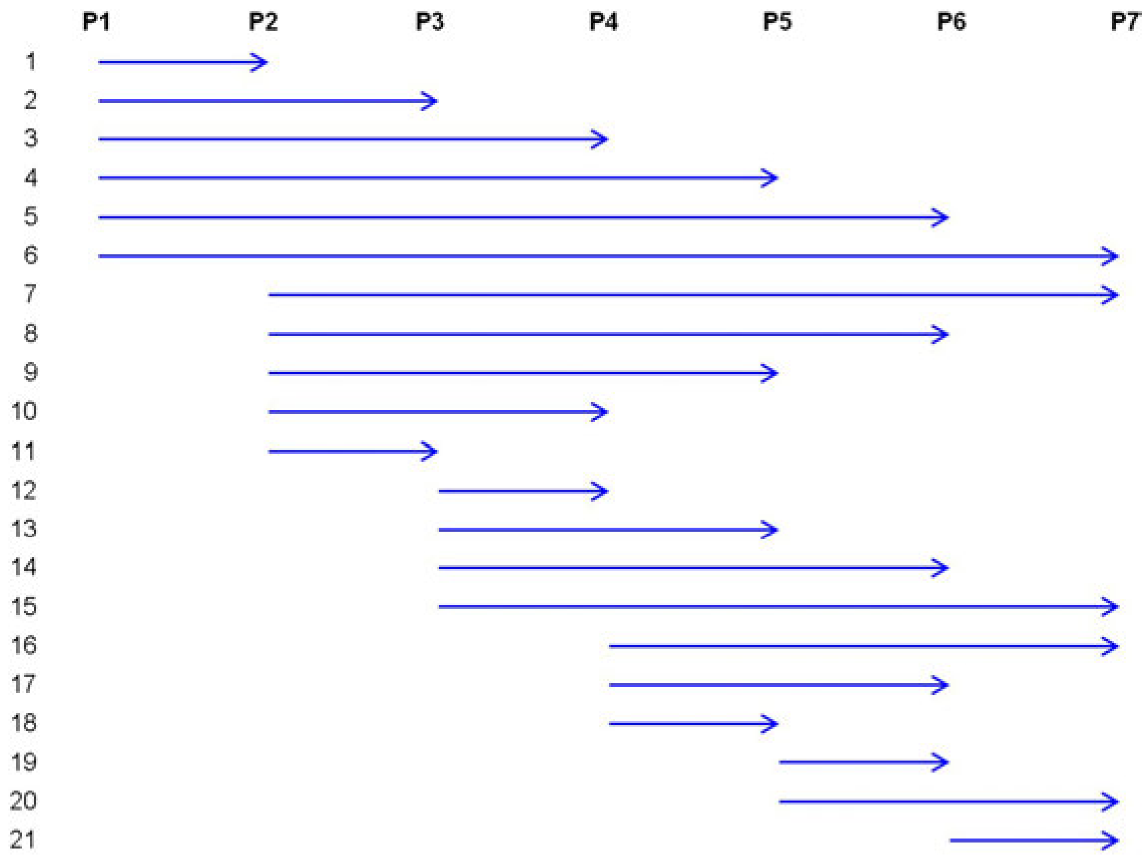

Figure 3: Observation sequence for 7-pillar baselines.

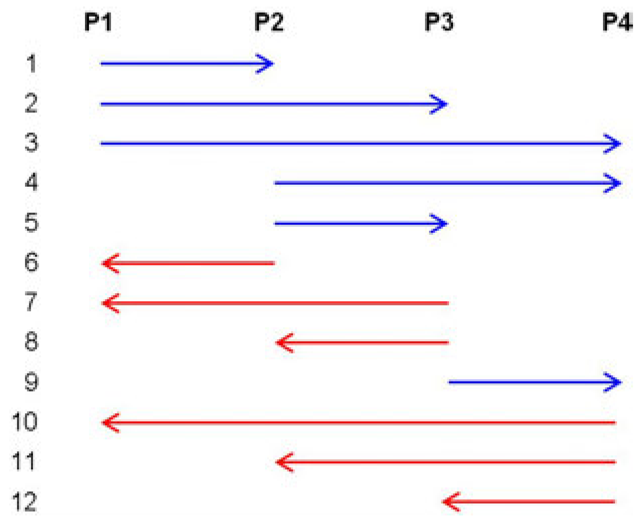

Figure 4: Observation sequence for 4-pillar baselines.

It is important to note that the accurate observation of meteorological data is essential for a reliable EDM calibration. An error in the measurement of 1ºC in temperature or 3 mb in atmospheric pressure will cause a corresponding error in the reduced distance of approximately 1 ppm. If possible, relative humidity (%) should also be observed (once for each inter-pillar distance), although it is recognised that its effect on EDM calibration results is minimal. Handheld met sensors currently available on the market are affordable and can provide temperature, atmospheric pressure and relative humidity in the one compact unit.

Upon completion of the field work, surveyors are required to restore any baseline security measures to their original state, e.g. replacing any protective caps and bolts securely to minimise damage to pillars caused by vandalism. Damaged or missing caps and bolts should be reported immediately to LPI, so repairs can be undertaken promptly.

5.3 Data Recording

All field notes and calculations relating to the EDM calibration are to be retained by the surveyor in order to maintain legal traceability of distance measurements. Records should be kept indefinitely because measurements made by any instrument (and at any point in time) may be questioned.

5.3.1 Manual Recording

It is strongly recommended to use the EDM calibration recording sheet available from LPI (2015a) for the recording of baseline observations in the field. All data entry fields should be completed. Although the recording sheet is self-explanatory, the following is a brief explanation of what details to record:

- Make, model and serial number of the instrument and the reflector used in the calibration.
- Make, serial number and correction to the thermometers and barometers used.
- Weather as it applies to the baseline, noting cloud cover, wind speed and direction, and the presence of heat shimmer, fog or rain if applicable.
- Pillar numbers in the ‘From’ and ‘To’ columns. Each pillar has a unique PM number fixed to the front of the pillar.
- Instrument height (H.I.) and reflector heights (H.R.) above the pillar plate, read to an accuracy of 1 mm.
- Temperature and atmospheric pressure as read. The correction to each reading is to be applied when reducing observations. It is advised to also record relative humidity.
- Slope distance measurements – the first column is for the entire distance and the following four columns are for the last two decimals only.
- Observations should be dated and signed by the observer.

In the event of booking errors, each mistake should be crossed out (not erased or made illegible) and the correct value entered alongside. All such alterations should be dated and signed or initialled by the person making the correction.

5.3.2 Electronic Recording

Although it is strongly recommended to use manual booking, it is acceptable to electronically record observations onto the instrument’s memory card. However, slope distances should be recorded in preference to the horizontal distances automatically reduced by the instrument. All other observations made including heights of the instrument and the reflector as well as meteorological data readings must be recorded as well – on the instrument’s memory card or elsewhere.

5.4 Data Processing

The data processing procedures should be commenced as soon as possible once field work has been completed. Firstly, the field observations must be checked against any electronic data recorded in the field. Booking sheets must be complete and include all mean calculations.

Once the raw EDM calibration data has been checked, it can be processed to determine the additive constant, scale factor and cyclic error (if required) using various software tools, e.g. EDMCAL (Janssen and Watson, 2014a; 2014b), the spreadsheet developed at the University of New South Wales (Harvey, 2014) or Baseline (Klinge, 2007). This section is included for those wishing to process the observations manually. It should be noted that the order in which the corrections are presented is generally the order in which they should be applied. The equations stated are based on those found in Rüeger (1996). For a detailed description of least squares adjustments and statistics related to surveying applications, the reader is referred to Harvey (2009).

5.4.1 Initial Processing

The following corrections are applied to the observed slope distances (d) in order to reduce these to 'horizontal' distances (d_{hz}) at the height of the lowest pillar:

$$d_{hz} = d + c_{atm} + c_{slope} + c_{height} \tag{1}$$

where c_{atm} = atmospheric correction
c_{slope} = slope correction
c_{height} = height (or datum) correction

The basic principle of EDM instruments is the indirect determination of the travel time of a wave of light from the instrument to the reflector and back. While the speed of light in a vacuum is well known, in practice measurements are (of course) not carried out in a vacuum. The EDM measurements must therefore be corrected for the ambient atmospheric conditions because the velocity of visible and infrared waves changes with temperature, atmospheric pressure and relative humidity (for light waves, humidity is often ignored). The atmospheric correction (also known as first velocity correction) c_{atm} is calculated as follows:

$$c_{atm} = \left[VC1 - \frac{VC2\, p}{(273.15 + t)} + \frac{11.27\, PWVP}{(273.15 + t)} \right] 10^{-6}\, d \tag{2}$$

where $VC1$ = reference refractive index specific to EDM instrument (also known as C)
$VC2$ = instrument pressure factor specific to EDM instrument (also known as D)
t = temperature (ºC)
p = atmospheric pressure (mb)
$PWVP$ = partial water vapour pressure (mb)
d = observed slope distance (m)

It should be noted that it is generally sufficient to adopt an approximate value of 15 mb for the partial water vapour pressure. The second velocity correction (accounting for the fact that the light wave does not follow a circular path between the two pillars) is more important for microwaves than for light waves and ignored for EDM calibrations because it is insignificant over such distances. Also insignificant over such distances is the reduction from observed wave path arc distance to wave path chord distance.

The slope correction reduces the slope distance to a horizontal distance at the mean elevation of the two pillars involved:

$$c_{slope} = -\frac{\Delta H^2}{2\,d} - \frac{\Delta H^4}{8\,d^3} - \frac{\Delta H^6}{16\,d^5} \tag{3}$$

where ΔH = height difference between instrument and reflector (m)

The height (or datum) correction (also known as sea level correction) reduces the horizontal distance at the mean elevation of the two pillars to the horizontal distance at the lowest pillar elevation:

$$c_{height} = -\frac{H_M}{R}\, d + \frac{H_M \Delta H^2}{2\, d\, R} + \frac{H_M \Delta H^4}{8\, d^3 R} + \frac{H_M \Delta H^6}{16\, d^5 R} \tag{4}$$

where H_M = mean height of instrument and reflector above the lowest pillar (m)
R = radius of curvature of the ellipsoid along the line (m), here assuming R = 6,370,100 m for New South Wales

Strictly speaking, this would be followed by the chord-to-arc correction, which converts the chord distance at the lowest pillar elevation to the datum arc distance. However, this correction is generally zero over distances used for EDM calibrations.

5.4.2 Solving for the Instrument Corrections

As stated previously, in order to achieve legal traceability of distance measurements all three instrument corrections (additive constant, scale factor and cyclic error) must be determined. However, for modern instruments the magnitude of the cyclic error is generally negligible.

The additive constant for a particular instrument/reflector pair is generally computed using the 'parts to the whole' method. As an example, a 4-pillar baseline would give the following combinations for the additive constant (AC):

$$\begin{aligned} 2\,AC_1 &= -\,(d_{12} + d_{23} + d_{34} - d_{14}) \\ AC_2 &= -\,(d_{12} + d_{23} - d_{13}) \\ AC_3 &= -\,(d_{23} + d_{34} - d_{24}) \\ AC_4 &= -\,(d_{13} + d_{34} - d_{14}) \\ AC_5 &= -\,(d_{12} + d_{24} - d_{14}) \end{aligned} \tag{5}$$

where d_{12} is the mean of the two reduced horizontal distances (forward and reverse) measured between pillar 1 and pillar 2, etc. The additive constant should be computed using a standard least squares adjustment as detailed in Rüeger (1996). In order to confirm that the additive constant has been determined correctly, the parts to the whole should be re-computed, using the corrected distances.

The scale factor is determined by computing the ratio of each of the inter-pillar distances (reduced to the horizontal at the height of the lowest pillar and corrected for additive constant) with the corresponding known distances shown on the baseline certificate. On a 4-pillar baseline, the scale factor is determined using the weighted mean of the six inter-pillar distances. However, an estimate of the scale factor can be determined by comparing the unweighted mean of three inter-pillar distances with the published values, e.g.:

$$\begin{aligned} \textit{scale factor} &= \textit{known distance} / \textit{observed distance} \\ &= 650.500 \text{ m} / 650.502 \text{ m} \\ &= 0.999\,9969 \\ &= -3.1 \text{ ppm} \end{aligned} \tag{6}$$

The cyclic error in modern EDM instruments is usually less than 2 mm in magnitude and can therefore generally be ignored. However, strictly speaking, it must be determined in order to achieve legal traceability of distance measurements. It should be noted that, if found to be significant, the cyclic error should be applied to the measured slope distances before the other instrument corrections are determined. All EDM baselines in NSW are designed to allow the cyclic error to be determined, and available EDM baseline software can generally be used in this regard.

The principle of determining the cyclic error can be explained using an example that does not require a baseline to be observed. In its simplest form, the test consists of laying out a calibrated tape horizontally along the top of a low wall for a distance corresponding to the unit length of the instrument (e.g. 10 m) at a distance of about 50-70 m from the EDM instrument. A reflector, mounted in a tribrach, is moved to each successive 1-metre (or ½-metre) graduation along the tape and both the EDM distance and the tape distance is recorded. The two sets of measurements are then compared (see Table 1 for an example). More details can be found in Rüeger (1996).

Table 1: Example of cyclic error determination using a calibrated tape measure.

Measurement	1	2	...	10	11
Calibrated Tape	2.000	3.000	...	11.000	12.000
EDM	50.125	51.126	...	59.124	60.123
Cyclic Error	**0.000**	**+0.001**	...	**-0.001**	**-0.002**

5.5 Analysis of the Calibration Results and Application of Instrument Corrections

The EDM calibration result should be checked to ensure that it reflects the quality of the instrument tested. As a general rule, the instrument correction should approximate the precision to which the instrument is capable of measuring distances, as stated by the manufacturer.

If the calibration result significantly exceeds the manufacturer's specifications, the following may have occurred and appropriate action should be considered:

- The instrument may not be in good working order and in need to be serviced and then re-tested.
- The observation procedures may not have been followed to a satisfactory standard, commonly caused by poor meteorological observations and/or low precision instruments, and taking shortcuts to save time.
- The verified baseline values may no longer be accurate. This is unlikely to occur if the baseline has recently been verified, but can occur if the baseline has been confirmed to be subject to pillar movement.

5.5.1 Cyclic Error

The cyclic error is generally insignificant in modern instruments and consequently not applied to the measured field distances. However, strictly speaking, its magnitude must be determined

in order to achieve legal traceability of distance measurements. If the cyclic error is found to be significant, it should be applied as a correction to the measured slope distances prior to reduction of the distances to the horizontal and the determination of additive constant and scale factor.

5.5.2 Additive Constant

Because the additive constant is determined without reference to the published inter-pillar distances, it is not influenced by changes in the true distances caused by pillar movements occurring since the baseline was last verified. The additive constant is primarily a correction for the combined physical offset of the reflector and the offset of the electrical centre of the instrument and, unlike the scale factor, should not be influenced by a change in the ambient temperature. Consequently, the additive constant should not vary significantly in subsequent calibrations, provided the same instrument/reflector combination is used.

The additive constant should be applied to all measured field distances either manually or by setting the constant in the instrument after the calibration. Once set in the instrument, a known distance should be re-measured to ensure the sign (positive or negative) of the constant has been correctly applied or set.

5.5.3 Scale Factor

The scale factor will generally vary for subsequent calibrations within the accuracy specification of the instrument because it is dependent on the instrument's modulation frequency, which may change with variations in the ambient temperature. To a lesser extent, the scale factor can also change as a result of frequency drift and ageing of the frequency oscillator. Consequently, if the scale factor falls within the instrument's specification, it should not be applied as a correction to measured field distances.

If the scale factor falls outside the instrument's specification, the instrument should be returned to the manufacturer for service. However, it is advisable to repeat the calibration under different climatic conditions both to confirm the result and to observe if the scale factor changes with different ambient temperatures. The thermometers and barometers used in the calibration should also be re-calibrated against a certified standard as an error in temperature and pressure readings will contribute to the scale error of measured distances.

5.5.4 Uncertainty of the Instrument Correction

The former National Standards Commission (now incorporated into NMI) recommended in 1983 that the minimum standard for the uncertainty of calibration of an EDM instrument should be 5 mm + 30 ppm at the 99% confidence interval (revised value from 1986 stated). This uncertainty is equivalent to 4 mm + 20 ppm at the 95% confidence interval and easily achieved with modern total stations. The uncertainty of the instrument correction (in relation to the national standard) includes the uncertainty of the verified baseline distances as shown in the measurement report for each baseline – currently the least uncertainty quoted for the verified inter-pillar distances is 0.5 mm + 1.3 ppm at the 95% confidence interval. The reader is referred to Rüeger (1984) for details on the calculation of the uncertainty of the instrument correction. The computation of this uncertainty, which varies with the distance range measured, is quite complex and therefore beyond the scope of this paper.

6 CONCLUDING REMARKS

The Surveying and Spatial Information Regulation 2012 requires that the length stated by a surveyor should not differ from the true value in terms of the State Primary Standard of measurement by more than 10 mm + 50 ppm at a confidence interval of 95% (NSW Legislation, 2014b). The required accuracy or uncertainty is to include the uncertainty of the length measurement arising from all possible sources. In addition to the uncertainty of calibration, length measurements made with an EDM instrument are subject to errors arising from the centring of the instrument and reflector, measurement of the atmospheric conditions and those associated with the reduction of the slope distance to the horizontal. A more detailed list of errors occurring in distance measurements with EDM instruments can be found in Rüeger (1996). It is therefore essential that all ancillary equipment is calibrated and in good adjustment and that an appropriate measuring technique be adopted in order to achieve the required result.

This paper has proposed updated best practice guidelines for EDM calibrations in NSW, which are expected to flow on into the next update of Surveyor General's Direction No. 5. It is important to note that calibrating an EDM instrument in prism mode does not calibrate the reflectorless EDM laser. These two modes generally have different additive constants and scale factors within any one instrument, i.e. testing in reflectorless mode must be performed separately – see Evans (2014) for more details. It should also be noted that if surveyors measure any distances that are longer than the longest line on their EDM calibration baseline, they should consider the reliability of the extrapolation of their calibration parameters.

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GNSS and the Professional Surveyor: A Practical Overview

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ABSTRACT

This paper provides a discussion on the use of Global Navigation Satellite System (GNSS) techniques in general and specifically for cadastral surveys. It is not a scientific investigation but presents a collection of observations and experiences gained by the author in the day-to-day use of GNSS equipment in a broad range of everyday survey projects. GNSS has been in commercial use since the early 1990s. In the last few years, hardware and software development has empowered those 'without skill in geometria' to enter the world of geodesy. Typically surveyors were the first to adopt new computer-based technology, including programmable calculators and computer-aided drafting (CAD), because of the high number of observations and calculations required to present a project. Today, with relatively cheap advanced software and technologically advanced equipment, we are seeing young engineers adopt software and GNSS as part of the Gen Y trip through life. This means the surveying profession is poised to lose a great deal of its traditional work. The use of static, Real Time Kinematic (RTK) and continuously operating reference stations (CORS) in daily applications is discussed in undertaking surveys in different types of applications. Differences between coordinates and vectors derived from GNSS observations are examined, considering that the cadastral world is one of vectors, while the GNSS world is one of coordinates. Einstein put forward the proposition that everything is relative. In the cadastral world, everything is relative to the ground or the monument. The GNSS observation is also relative – relative to the base station and the last observation made. It is shown that, in the world of cadastral surveying, GNSS is an ideal tool with which to re-establish a rural cadastral boundary.

KEYWORDS: *GPS, GNSS, cadastral surveying, rural boundaries.*

1 INTRODUCTION

The term Global Positioning System (GPS) is the most common term used by the general public to describe any technology associated with satellite-based position fixing. This is derived from the constellation of U.S. satellites commissioned in 1993. Considering similar systems originating in other countries (e.g. Russia's GLONASS, China's BeiDou and Europe's Galileo), these are known as Global Navigation Satellite Systems (GNSS). In recent years, the third generation of GNSS includes the use of mobile phone technology.

The surveying profession has always been at the cutting edge in adopting new technology for data processing. The large amounts of data surveyors collect on a daily basis and the relative complexity of processing this data has prompted surveyors to be the first and most enthusiastic adopters of computer and software technology. The introduction of the New South Wales Integrated Survey Grid (ISG) in the 1970s was incomprehensible to the general public and complicated for the general surveying profession (Lands, 1976). We have now adopted the Geocentric Datum of Australia 1994 (GDA94) and its Map Grid of Australia

(MGA) projected coordinates as the standard (ICSM, 2009). The MGA is the latest incarnation of a Universal Transverse Mercator (UTM) grid system for Australia and is indispensable to the operation of any large-scale survey projects such as highways and railways. For a review of coordinate systems, datums and related transformations, the reader is referred to Janssen (2009).

In 1980, the only people who could undertake any work on the ISG were specially trained university graduates. Today, technology has enabled ‘the man on the Clapham omnibus’ to undertake any number of measurements and calculations using GNSS technology. It is at this point in time that the paradigm of the professional surveyor has changed forever. Today, with relatively cheap advanced software and technologically advanced equipment, young engineers adopt software and GNSS as part of the Gen Y trip through life. This means the surveying profession is poised to lose a great deal of its traditional work.

The surveying profession consists of a collection of university graduates trained and competent in the skill of geometria. Today, there is a definite erosion of the profession’s monopoly in geometria. This paper introduces a range of issues in using GNSS in private practice, particularly in cadastral surveying.

2 THE PARADIGM

2.1 Available GNSS Methods

Reference to Surveyor General’s Direction No. 9: GNSS for Cadastral Surveys (LPI, 2014) notes the accepted methods of GNSS as being:

- static
- static using Continuously Operating Reference Stations (CORS)
- AUSPOS
- Real Time Kinematic (RTK)
- RTK or Network RTK (NRTK) using CORS
- Precise Point Positioning (PPP)

2.2 Professional Users

Over the last few years, the author had the opportunity to speak at several surveying gatherings on the use of GNSS in one form or another. It was surprising to see a general lack of knowledge in the profession on the use of GNSS for different types of surveys. There seem to be a number of camps:

- Those who are competent and knowledgeable.
- Those who use it well.
- Those who are cowboys (these have some knowledge but take every shortcut possible).
- The plainly incompetent.
- The doomsayers.
- The non-interested.

2.3 New Users

Today a new set of users is adopting GNSS technology with significant impacts on the professional surveyor. GNSS technology is now being adopted by:

- Younger civil engineers.
- Local government authorities.
- Construction companies.
- Mining companies.

2.4 Some Examples

It has been the author's experience to date that, with the possible exception of mining companies, none of these users have the required level of knowledge in surveying or the consequences of using GNSS. Some years ago at an Australia Day Conference a surveyor made a presentation in which they stated "GPS is great, we employ farmers and pay them $12 an hour to do our surveys." This presentation incited either great mirth or anger in the audience. The interesting thing is that this surveyor was predicting the future. Nowadays GPS units are affordable and useable by the non-surveyor. The commissioning and ongoing expansion of the CORSnet-NSW network of permanent GNSS reference stations across NSW (e.g. Janssen, 2014; LPI, 2015a) has further encouraged this uptake because the user does not have to use their own base station. It could be argued that some product salesmen are overselling the equipment to those who are untrained in its use. A typical response when querying untrained users on their methodologies is "That's what the salesman told me to do."

3 WHAT IS THE DATUM?

The best feature of GNSS is that you always have the same azimuth. This can bring conflict and confusion in some instances where there are distortions in the ground control marks contained in the Survey Control Information Management System (SCIMS – see LPI, 2015b). There are inconsistencies within the SCIMS network due to the hierarchy of various adjustments. These are probably more pronounced in rural areas because of the greater distances.

The SCIMS network is based on a series of adjustments using many thousands of observations. In the 1990s, the NSW Government undertook intensification in the network, resulting in many adjustments over a short period of time. CORSnet-NSW is based on a different realisation of the Australian datum, i.e. the SCIMS coordinates and the coordinates used in CORSnet-NSW are different (Janssen and McElroy, 2010).

In cadastral surveying everything is relative to a monument. If boundary definition remains based on common law, the surveyor is more interested in the monuments and vectors connecting these monuments.

3.1 Localisation

The term even causes confusion and argument. This term should not be confused with validation, see section 4 of the Surveyor General's Direction No. 9 (LPI, 2014). A localisation (or better site transformation) is undertaken by observing one or a greater number of points and allowing the GNSS unit to adjust its observations to the ground control (Haasdyk and

Janssen, 2012). In the case of multiple points, this effectively distorts the observations to fit the observations into the existing fabric of surrounding control. From a practical point of view, it is not possible to repeat a localisation as there will always be some differences in the derived coordinates. Therefore if there is an error in a localisation, that error will be spread throughout the job and the next surveyor may not agree with the measurements.

Our practice is to ban localisations (except for machine control) and we now refuse to help other people (both surveyors and non-surveyors) who have problems because a localisation was undertaken.

3.2 What Should Be Adopted?

This is a question for all surveyors to ponder rather than for the author to answer.

3.2.1 Case Study 1

As with any survey measurement, there is a quantum of uncertainty in measurements derived from GNSS observations. Surveyor General's Direction No. 9 states that for rural surveys, CORS RTK is acceptable.

When undertaking large control surveys for aerial purposes, we often use a combination of GNSS techniques. The process is often a combination of static and RTK observations. We will undertake RTK with two rover units set at different heights, i.e. one rover on a tripod and the other on a fixed height pole. This gives an independent check on height, one fixed pole height and one variable (measured) at each station. If phone reception is available, we also use CORS RTK. Table 1 shows a summary of part of a dataset.

The interesting points to note are:

- Point numbering.
  - o points numbered 3xx have been taken to a mark using RTK
  - o points numbered 5xx have been taken to the same mark using CORS RTK
- Point 501 is a star picket established on top of a hill with good skyview.
  - o this was measured using CORS RTK and adopted for the field survey as the base station for other RTK measurements
- Point 1 shows the coordinates calculated from a set of 8 hours observations based on SCIMS A1 control approximately 20 km away and CORS static as check calculations.
- Points 355 & 555 are the same witness mark in close proximity to the base station measured using RTK and CORS RTK.
- Points 310 & 510 and 312 & 512 and 314 & 514 are points of interest measured using RTK and CORS RTK.

Our use of CORS RTK in this case was for gross error detection. The four points were chosen for good skyview and lack of interference. In this case, it would appear that the differences are due to elevation. The horizontal coordinate differences are significant but the vectors in the case of the last three points are acceptably close.

Table 1: CORS and RTK comparison.

Point	Easting (m)	Northing (m)	AHD71 Height	Comment
1	282619.930	6508662.196	645.411	8 hours static from SCIMS
501	282619.935	6508662.177	645.396	CORS & adopted for RTK field base
Static-CORS	-0.005	0.019	0.015	
The base station was given the field coordinates of the CORS observations as this would allow quick verification in the field. The CORS base station is approximately 50 km away. The calculated and observed coordinates are remarkably close. The height difference in this case may be considered as a control.				
355	282617.810	6508660.035	645.262	RTK witness mark
555	282617.817	6508660.031	645.219	CORS witness mark
RTK-CORS	-0.007	0.004	0.043	
Similar differences are evident in the Easting and Northing. In this case the height difference is an indication of the delta height attributable to the observations using RTK and CORS RTK.				
310	281594.051	6508631.324	453.024	RTK (1.02 km to base)
510	281594.089	6508631.489	452.980	CORS
RTK-CORS	-0.038	-0.165	0.044	
This point is about 1.02 km from the base station to the south-west. This is the first point of interest and the delta values have changed considerably.				
312	282286.509	6510655.614	441.861	RTK (2.02 km to base)
512	282286.577	6510655.780	441.924	CORS
RTK-CORS	-0.068	-0.166	-0.063	
This point is about 2.02 km from the base station to the north-west. The delta Easting and Northing values are consistent with those of point 310/510 above.				
314	282292.042	6510659.187	441.569	RTK
514	282292.108	6510659.342	441.606	CORS
RTK-CORS	-0.066	-0.155	-0.037	
This point is in close proximity to point 312/512. The delta Easting and Northing values are consistent with those of point 310/510 and 315/514 above.				

3.2.2 Case Study 2

This job was to replace a Permanent Mark (PM) that would be destroyed by Council road works. In this case, SS34224 (Class A Order 1) and PM31456 (Class C Order U) were chosen as the datum of the survey. The new mark (PM182136) was installed in close proximity to PM31456 to maintain the inter-visibility of the existing marks.

In this case, a conventional traverse was undertaken and a series of observations using the nearby CORS station were undertaken on the tripods at the same time as the terrestrial survey thus eliminating centring errors.

The three observations stated above the line in Figure 1 are the point of comment:

- SCIMS ground was used for azimuth.
- CORS RTK is the same.
- The ground distances are the more interesting as the CORS RTK and Electronic Distance Measurement (EDM) observations agree with each other but are significantly different to the SCIMS ground distance.

The CORS RTK coordinates (after adjustment to the SCIMS network for the CORS station) differed to the coordinates in SCIMS.

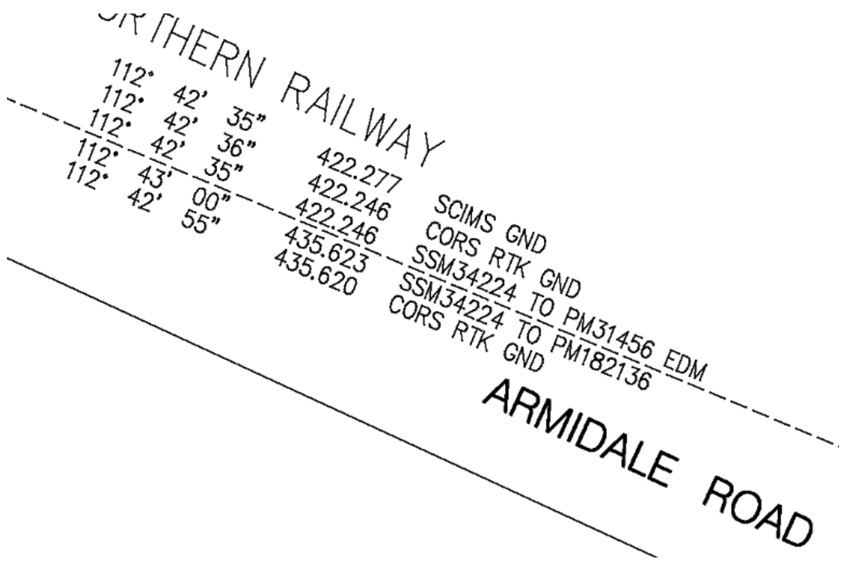

Figure 1: CORS RTK and traverse lines.

3.3 What Is Practical?

When undertaking a cadastral survey in a rural area, the author does not yet have the confidence to use CORS RTK. I do believe that CORS static is a very good solution where there is a dearth of coordinated SCIMS marks. Currently, we use RTK with connections to either the SCIMS network or to our own CORS static observations. We also like to use CORS RTK to track the usability of CORS RTK in a rural environment. A practical approach is to set up the job in MGA 56 and AUSGeoid09 set as the default in the GNSS unit. The azimuth will be consistent.

4 WHAT ARE THE TRAPS?

There are many opportunities for making mistakes when using GNSS to undertake any type of survey. It is especially important to be conversant with the operation of the equipment one is using. Each brand has its idiosyncrasies. There are settings that allow the operator to achieve higher accuracies. There is a tendency for users to set the equipment to instantaneous and move as fast as possible. The time taken making a couple of minutes of observations can be utilised to make good field notes.

4.1 Some Tips

4.1.1 Base Station

Choose the site for the base station carefully. The base station is the most important point in the job. Anything that interferes with the efficiency of the base station will affect every other point in the job.

- Avoid stations near trees.
- Avoid power lines and transformers where possible.
    - If the only coordinated marks are near trees or a power line, set up a base station in a suitable place and observe back to the SCIMS marks.
    - Use a theodolite and traverse into the control marks if they are under trees.
        - It is easy to get a good long azimuth with a GNSS unit.

- If working in a hilly area, establish a base station on the top of a clear hill.
- Use an aerial on the radio that suits the environment, broomstick aerials are good on the plains but do not perform well in the mountains.
- Always use a height for the base station, even if it is derived by the GNSS unit as a ‘here’ point.
- Measure the height of the base station and use the height of the witness mark as a check. Heights are the most sensitive indicator of a problem.
- Establish a witness mark near the base station.
    - Check onto the witness mark at the beginning and end of each job.
        - Record your observations.
    - Check onto the witness mark at any available opportunity during the survey.
    - The witness mark is your personal verification.

4.1.2 Rover

- Use a tripod to measure points.
- Pogo sticks and other adapters are often useful for measuring non-ground points.
- If using a pogo stick, it is best practice to use a bipod.
- If using a pogo stick to take measurements, be aware of the error due to the height and the sensitivity of the bubble.
    - Take four measurements and rotate the receiver by 90º for each measurement.
- Take a number of measurements at a point.
    - Download the points and look at the spread of coordinates.
        - Take four measurements. This allows one to be discarded with three still remaining.
    - Have the instrument set so you can choose not to accept a measurement. Experience has shown that discarding the first measurement as a general practice seems to group the subsequent measurements more closely.
- The author concludes that taking four measurements at a 30-second epoch is a more reliable methodology that taking one measurement of 120 seconds.

Figure 2 shows a rover with an adaptor used for measuring posts and fences. In this case, the mile post is on the side of a hill that is almost impossible to walk along. We also took a measurement at the base of the post as it had a significant lean. In addition:
- Establish pairs of marks, so the survey can be infilled with a theodolite.
- Measure between these pairs of marks as a form of continuous verification.

4.1.3 Validation

Measuring a point twice at a different time of the day is not a difficult task on a rural job. Using the EDM to check measurements at opportune times is also a good validation practice.

Figure 2: Adaptor for measuring posts.

5 A PRACTICAL APPROACH

There are a number of methodologies for undertaking a rural cadastral survey with GNSS. It is probably more important to be aware of the methodologies and use the most appropriate method in any particular circumstance.

The least favoured methodology is ‘radiation sickness’ (a non-academic term coined by Henry Werner). This entails loading all the data in the GNSS and looking for the points. This methodology is generally favoured by the less experienced. A clever trick is to load this data as a separate job and you can easily extract vectors between points in the field.

The author’s preferred method is to use MGA and scale ground distances and rotate magnetic bearings to MGA grid. This is exactly the same methodology we used to use when we measured with a wire and did our calculations on a HPxx. The most important thing is to establish a base station in a good location. If no suitable SCIMS marks are available, choose either ‘here’ on the GNSS unit or, if phone coverage is available, take a CORS RTK reading.

5.1 How To Obtain An Azimuth Comparison

Obviously, the easiest way to achieve an azimuth comparison is to find two marks on the one plan. This is not always as easily done as it is said. A good trick is to take a measurement on a point that can be seen from some distance, e.g. the top of a fence post is a good target. Walk a good distance from the post, take a GNSS observation, and now make an observation to the post with your compass.

If you are going to an area where it may be difficult to find survey marks, check the photography on Spatial Information Exchange (SIX – see LPI, 2015c) maps. Locate a fence

on SIX maps that may be on the boundary and calculate the bearing from the photography on SIX maps. You now have an approximate MGA position from SIX maps and a magnetic bearing from the plan.

The mile post shown in Figure 2 was located on the second day of the survey by using a combination of these methodologies. Inspection of SIX maps showed no fencing approximating the boundaries in or around the job. The azimuth comparison was taken from SIX maps from a fence about 5 km from the job. The cadastral overlay was examined and a couple of trees were chosen that may have been reference trees. The data was swung onto MGA and a block shift undertaken to get an approximate location. On the first field day, a reference tree was located and some measurements taken around a bend in the creek. The data was then shifted to suit these observations. On the second day, the approximate location of some of the boundaries was walked using the rover. It was on this day that some other reference trees, the mile post and some remains of old fencing were located.

6 CONCLUDING REMARKS

Most of the author’s GNSS experience has been gained not in the cadastral world but in areas such as large-scale control for photogrammetry and LiDAR, working in the mines, mining exploration, civil construction and conducting detail surveys. Using a GNSS for up to 10 hours a day gives the user valuable insight into the vagaries of the equipment. For example, on one job we realised there was a ‘hole’ in the satellites at about 11:00 hrs EST. After a few days, we noticed the accuracies returned about half an hour later. Over the course of the job we watched the ‘hole’ move a little each day.

Sometimes a site can pose a particular problem. One site was most difficult as the GNSS unit failed to perform regularly. The author has not been able to determine the cause of the problem except that it was localised. A colleague had a similar situation – in that case it was traced to the intercom system in a nearby gaol interfering with the radio.

GNSS in the right hands is a wonderful tool. Its use for rural cadastral surveying is far superior to the traditional terrestrial methods. In one case using the practical approach outlined above the author found a number of stone locks pits where there was no fencing within close proximity of the boundaries. Locating two of the original marks allowed scaling and rotating the data to MGA. It was then possible to walk the boundary and subsequently find a number of stone locks pits (in the long grass) along the boundary line.

GNSS users should not become obsessed with accuracy. Modern GNSS is accurate and repeatable, and users should develop good field, office and checking techniques. Then you will re-establish cadastral rural boundaries in accordance with the principles of boundary surveying and common law.

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Staff Surveyors' Association: A History

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ABSTRACT

The NSW Staff Surveyors' Association (SSA) held its first Annual Conference in 1921 with the objective of looking after the professional and vocational interests of Surveyors employed in the NSW Department of Lands. The SSA surveyors were widely scattered and isolated throughout New South Wales, and initially this Association was their only mechanism for professional development and collective representation. This paper tells the story of the significant events of SSA since its inauguration. This information was obtained by going through published papers and proceedings, combing through minutes of meetings and by holding interviews with some of the past members of SSA. It also includes the author's own recollections and interpretation of events. The SSA was instrumental in the formation of the Association of Public Authority Surveyors (APAS), almost 80 years later. This paper includes a description of the events that led to the formation of APAS and the drafting of its constitution. The main objective of this paper is to record the history of the SSA. It will also showcase how organisations that are alert to their environment can identify necessary changes, take bold initiatives and adapt to the times. This paper proves that the future is clearer by acknowledging past events.

KEYWORDS: *Staff Surveyors' Association (SSA), history, Association of Public Authority Surveyors (APAS), professional development, unionism.*

1 INTRODUCTION

The Staff Surveyors' Association (SSA) was the body responsible for the formation of the Association of Public Authority Surveyors (APAS). It has held 90 conferences since 1921 and achieved many great things for its members, and also contributed generously to the orderly economic and physical development of New South Wales (NSW). This paper records a few significant events of the SSA and gives an insight into the rich heritage that APAS has inherited.

The story of the SSA has its roots in the first (British) settlement of Australia. Staff surveyors worked in isolation in regional centres with little contact with their professional brethren. It was only when the surveyors employed in the Department of Lands formed the Staff Surveyors' Association, did they get adequate training and gain a collective voice. Since its formation, the Association has helped shape the work of surveyors in the Department of Lands for the benefit of the rest of the State.

In 1993, SSA decided to spawn an organisation that was more relevant to the times and proposed the formation of an Association of Government Employed Surveyors – later to be called APAS. In 2014, the members of SSA decided to disband the Association, as APAS is

an excellent instructional mechanism, and the Public Service Association (PSA) fulfils the members' vocational representational needs.

This paper is based on information obtained from published papers and proceedings, minutes of meetings and interviews with some of the past members of the Association. It also includes some of the author's own recollections and interpretation of events. Retired Surveyor General and Past President H.E.H. (Harry) Barr presented a paper on "The history of the NSW Staff Surveyors' Association" at the 50th Anniversary Staff Surveyors' Conference in 1971 (actually the 47th conference). Unfortunately, only a brief transcript of his presentation was published in the proceedings, and the full written paper was not found for this paper (SSA Conf. 1971, p.34).

In fact, complete records of the Association were not found for the preparation of this paper, and accordingly there are some omissions, and some of the facts are only reported in general terms.

2 HISTORY OF STAFF SURVEYORS

The history of the Staff Surveyors' Association is interwoven with that of its parent organisation, the NSW Department of Lands. Telling the story of the SSA is to some extent telling the story of the surveying function within the Department of Lands and its successors, such as, currently, Land and Property Information (LPI).

2.1 From Colonisation to 1920

It was the Surveyor General's responsibility to turn unmeasured land into a saleable commodity. Surveyors and assistant surveyors of the Surveyor General undertook this work. They were gentlemen, professionals and well educated. Their salary was well above that of clerks. However, as time passed, there was a lack of acknowledgement of the role of surveyors in the opening up of the colony. Also, there was always too much survey work and not enough men and resources. Furthermore, the then Surveyor General Thomas Mitchell embarked on the trigonometrical survey of the known colony, drawing away surveyors from the pressing local and cadastral survey work developing the colony.

In order to enable more surveyors to undertake the work, in 1837, the first Board of Surveyors was established to examine surveyors for their professional competence (Kass, 2008, p.27). Surveyors did not receive formal education; instead they worked a form of unpaid apprenticeship in the Surveyor General's Office for some months.

In 1853, to improve efficiency, the Department of Lands created the regional Crown Lands offices structure (Kass, 2008, p.34). This was an attempt to deal with the enormous backlog of survey work for settlement of the colony.

Then in 1856, New South Wales achieved responsible government. This had profound implications for public servants in NSW. Newly elected colonial politicians were anxious to reduce the power and salaries of the large public service bequeathed to them by the imperial government. Highly paid elite bureaucrats were pensioned off – although (initially) surveyors were relatively unscathed, as their skills were in very high demand.

In order to enable coordination of the regionally based survey workforce, the Surveyor General produced the ‘Surveyors Pocket Book’ in 1886. However, according to Surveyor General P.F. Adams, these textbooks did not have enough information about all matters that surveyors needed to know. It included some advice on calculations, tables of calculations, as well as notes on the use of the newer survey equipment, especially the steel riband (Figure 1). So, for that time, surveyors in the field laboured measuring large areas of land, with only limited training.

Figure 1: Steel riband.

The Institution of Surveyors was formed in 1881 under the Institution of Surveyors Incorporation Act. Surveyor General P.F. Adams felt that surveyors “were not advanced enough” in the colony to New South Wales to need an Institute. He said that surveyors in his Department had little interest in the Institute, as it did not discuss the scientific matters that were of interest to them (Kass, 2008, p.140). The initiative to form the Institution of Surveyors New South Wales came not from the ranks of the Lands Department surveyors, but from the surveyors in private practice.

Then in the 1890s, staff surveyors were leaving the public service to make a more lucrative living in the private sector. To undertake the pressing survey work at that time, the District Surveyors had two salaried surveyors and several contract surveyors. The salaried surveyors were classified by length of service for all matters, which also included valuations, appraisal of rents, and inspection of contract surveys. Surveyors were under pressure.

In 1900, the Institution of Surveyors issued a strong statement about the need for both adequate remuneration and status for surveyors (sic) (Kass, 2008, p.167). Until this time, the surveyors were “nabobs” in rural areas, and instrumental in the orderly settlement of the colony – linked to its wealth and prosperity. Staff surveyors undertook arduous work, were highly regarded, but underappreciated.

Following the introduction of the Crown Lands Amendment Act, where more lands were made available for settlement, field surveys were impeded by the acute shortage of staff surveyors. The Minister of Lands sought to attract suitable men to be trained by the Department (Sydney Morning Herald, 8 Aug 1907, p.6). Also affecting the performance of surveyors in the Department was in 1916 the introduction of a bill to register surveyors. It was nearly two decades before the bill finally passed into legislation.

2.2 From 1920s to WW2

The Staff Surveyors’ Association was formed in 1920, with 56 members. By March 1921, it had 68 members, i.e. all the eligible staff surveyors employed by the Department of Lands except for three who refused to join (SSA Conf. 1921, p.4).

The first Annual Conference was held on 29 March 1921. The SSA provided the professional training needs of rural surveyors to carry out their duties. In addition, they had the opportunity to learn about their administrative duties within the Department of Lands. It became a major meeting place where the staff could learn from each other and cement professional friendships (SSA Conf. 1921, p.13).

The Association also set out to represent the surveyors' industrial interests and, to that end, affiliated with the Professional Officers Association. In 1926, the then President of the Public Service Association (PSA), Mr J. Brown, courted the Association with an address made at the Annual Conference (SSA Conf. 1926, p.2). The Proceedings of the 6th Annual Conference indicated on its title page that it was "affiliated with the New South Wales Service Professional Officers' Association". On the title page of the 7th Annual Conference in 1927 it showed "sub-section of Public Service Association". Arrangements were decided between the Public Service Association and the SSA by a referendum of all members "by a fairly small majority (25 to 21)" (SSA Conf. 1927, p.15). This relationship with the PSA continues to this day.

The Association started communicating with the Institution of Surveyors with the objective of raising the status of public service surveyors. Until that time it was their view that the Institution only represented private surveyors. On the title page of the 7th Annual Conference in 1927 it showed "incorporated with the Institution of Surveyors New South Wales". This is the first indication that a relationship had been established with the Institution, and the Association was entitled to have two representatives to the Council of the Institution.

Staff surveyors were under enormous pressure of work both during the Depression and the post-war period when the closer settlement of NSW was the big issue. Poor wages and the efficient use of vehicles characterised the industrial interests of the early period of the Association.

Higher standards and greater governmental control of private surveyors were implemented at about this time. On 7 January 1927, uniform regulations for the registration of surveyors were gazetted. The Surveyors Act 1929 was passed, and the Surveyor General was appointed President of the Board of Surveyors. However, the main issue dealt with by the Association that year was that of salaries. It was observed, though, by the President of the Association, Mr J.S. Turner, that it was the Association providing further education for staff surveyors – not the Department of Lands (SSA Conf. 1927, p.40).

The SSA was instrumental in introducing the motor vehicle into use for surveying (Figure 2). The Association "anxiously pressed" that a more equitable allowance be paid for the use of a car in their work. A sub-committee of the Public Service Board plus the Surveyor General was appointed in 1925 to consider making motorcars part of the equipment of surveyors. "It recommended the gradual replacement of horse equipment by motorcars" (Lands Annual Report 1925, p.19). In 1929, vehicle manufacturers advertised in the SSA Conference Papers and subsidised the printing of conference papers (SSA Conf. 1929, p.11).

It was the Depression that caused the Staff Surveyors' Conferences of 1931 and 1933 to be cancelled. At that time, the Public Service Board review sought to remove the position of Senior Surveyor in 1932, even though demand for their services increased. The Lands Department reduced its staff by 92 due to the Depression from 1927 to 1931. However, demand for surveyors' time and skill increased. Surveyor General Mathews implored the staff

surveyors "to exert themselves even more to meet the demands of their work" (SSA Conf. 1932, p.3; 1934, p.2).

Figure 2: Parliamentary inspection of TS2761 Kosciuszko in the 1920s.

During the Depression of the 1930s, there was a lot of work, large numbers of reappraisals of soldier settlement blocks. However, Treasury did not provide additional staff assistance. Surveyors were overworked to the point that some became ill. This was a time of retrenchments and staff surveyors were reduced in numbers.

Staff surveyors responded by training at conferences, exemplary performances in the field and courts giving praise from Surveyor General Mathews in 1937: "You have done your job well – dealt with £20,000,000 worth of property – and without one breath of suspicion" (SSA Conf. 1937, p.6). However, there was a heavy price to pay: "There was illness and breakdowns" (SSA Conf. 1935, p.3), leading Surveyor General Mathews to implore his surveyors to ease up – "It is possible to go too far in giving service" (SSA Conf. 1934, p.22).

With the spread of Sydney, demand for quarter and half acre housing blocks for the unemployed increased for land unsuitable for agriculture. By 1935, demand for residential land revived, providing survey activity for staff surveyors (Kass, 2008, p.260).

2.3 From War Years to 1970s

The Lands Department commenced WW2 with reduced staff after budget cuts of the 1930s. The staff surveyors were poorly prepared for the stresses that were placed upon them during the War. Staff Surveyor numbers progressively fell so that remaining staff had to work even more strenuously. The war had a big impact on the Staff Surveyors' Association, causing it to abandon conferences in 1942 and 1943 (SSA Conf. 1944, p.16). Department of Lands surveyors' work shifted from developmental to more strategic.

Surveyor General A.M. Allen was fundamental in enlarging the role of the Lands Department surveyors with the Emergency Mapping Scheme. Four survey parties were taken from the Lands Department survey staff for military mapping. The President of the Association welcomed the withdrawal of the staff for this work: "There are definite signs that they [surveyors] were to have their experience and capacity recognised, and made full use of in the service of the State" (SSA Conf. 1941, p.5).

In 1944, the SSA passed a resolution "calling for a contour survey of the State as soon as possible" (SSA Conf. 1944, p.22). However, the Under Secretary responded that "this could not be undertaken until some of the backlog of work and other pressing matters were in hand." By 1944, the survey staff of the Department of Lands had been depleted by:

- Staff economies of the depression era.
- The absence of men in military service.
- The fact that no new appointments had been made for the past 10 years.

During these times, staff surveyors felt much undervalued. They were appreciated for their professional abilities and hard work. The bulk of survey staff was over 40 years old. In 1946, Senior Surveyors had an average age of 55 years old (SSA Conf. 1946, p.22). Indeed, with the few appointments, it seemed that departmental policy was “aimed at the extermination of the breed” (SSA Conf. 1944, p. 11).

The unsatisfactory salaries and the staff shortages were taken seriously by the Lands Department. In 1946, the Minister of Lands made representation to the Public Service Board to raise the salary levels of surveyors. Through the untiring work of the Association, by 1947 staff surveyors were more satisfied and restored to their rightful importance in the Department and their salaries improved (SSA Conf. 1947, p.5). All District Surveyor positions were finally given permanent positions (SSA Conf. 1948, p.5). The Lands Department undertook an active recruitment, and in 1948 there were 43 staff surveyors (SSA Conf. 1948, p.24-25).

The influx of young surveyors from the armed forces strengthened the Department. The new recruits faced rapid promotion, since it was estimated that within 15 years every District Surveyor and Senior Surveyor would have retired. In 1950, it was reported that the survey staff was “nearing a full panel of field assistants, and there were quite a number of pupil assistants in training” (SSA Conf. 1950, p.13).

The Association even influenced the shape of the university course that was developed for surveyors, as it was widely canvassed at SSA Conferences. A wide range of ‘non-survey’ subjects was included in the course, which met with a good deal of opposition before content was finalised (SSA Conf. 1949, p.8). Eventually, topics such as land utilisation, aerial photography and land valuation were included.

In June 1947, the Central Mapping Authority (CMA) was formed and the Assistant Director of Mapping Mr Middleton was appointed under the supervision of the Surveyor General. However, trigonometrical survey work was still suspended in 1948 due to the desperate need for War Service Land Settlement surveys. The CMA became a properly functioning unit in 1951 when equipment ordered from overseas finally arrived in Australia.

In 1961, membership rules were amended to include not only registered surveyors, but also all surveyors graded by the Public Service Board as surveyors, whether permanent or temporary. This enabled all trigonometrical surveyors employed in the Department of Lands to be full members. Associate members (who did not have voting rights) were also able to join the Association, which included assistant surveyors, pupil assistants and trainees in the Surveying degree course (SSA Conf. 1961, p.9).

Full-time trainees in Surveying at the University of New South Wales (UNSW) sponsored by the Department of Lands were invited by the Association to attend the papers presented at the conference (SSA Conf. 1961, p.3). The number of attendees at the SSA’s Annual Conference swelled from that time because assistant surveyors were granted special leave and rail warrants to attend.

In 1969, it was reported that the Surveyor General had a full complement of staff with every position filled (SSA Conf. 1969). Together with the formation of the CMA and recruitment, the 1970s were a time of growth.

2.4 From 1970s to Formation of APAS in 1993

The Staff Surveyors' Association celebrated its 50th year of operation at its 47th conference in 1971. The membership of the Association had swelled to the highest numbers ever because of the recruitment of surveyors to the Topographical Mapping Program and the creation of the CMA. Another reason for the greater membership was the commencement of the Department of Lands 'cadetship' scheme that sponsored university students through the surveying degree, with the view of further employment as surveyors. From December 1969 onwards, all surveying candidates needed a degree from UNSW. Membership records show that there were 21 students in 1972.

Another influence on the membership numbers of the Association was the Homesite Development Program. The Department of Lands was the largest landholder in the metropolitan area. In 1973, the Surveyor General was a member of the State Planning Authority in recognition of the Department's large land development role. So in the late 1960s and early 1970s, surveyors were not only laying out subdivisions, surveying and overseeing road construction, but also completing trigonometrical surveys and control surveys for a standard mapping program. Accordingly, the category of cartographic surveyor was introduced to undertake this work, which also qualified as a full member of the Association.

One of the issues discussed at length at the 1974 Annual Conference was the appointment of the CMA Director H. Rassaby. The CMA was torn away from the Surveyor General's responsibility, along with the decision to create a separate authority and relocate to Bathurst (SSA Conf. 1974, p.59). Mr Rassaby famously announced that with the relocation to Bathurst, "employees will find that they have time for recreational activities and mapping will be a hobby" (SSA Conf. 1975, p.52).

In 1978, the SSA held its conference in Bathurst, the first conference outside of the CBD of Sydney. In 1979, the official adoption of the flexitime agreement was successfully negotiated, taking into account the fact that surveyors worked much of their time "in the field". This was the introduction of the 35-hour week for surveyors (SSA Conf. 1979, p.7).

During the mid 1980s, the Association was actively involved with the PSA in the campaign to preserve the benefits of the State Superannuation Scheme. However, the campaign did not stop the government of the day ending the scheme.

In 1993, SSA decided to spawn an organisation that was more relevant to the times and proposed the formation of an Association of Government Employed Surveyors (later to be called the Association of Public Authority Surveyors), and APAS was born (see section 3).

2.5 1993 to Present

Following the formation of APAS, the Staff Surveyors' Association continued principally with its industrial function. APAS took the lead in providing continuing education and the presentation and publication of papers.

The Department of Lands went through organisational turmoil during this period, where even management did not know where its survey function belonged. The Association dealt with many issues relating to the changing structure of the Department of Lands: Department of Information & Technology Management, Land and Property Information, and even the Department of the Surveyor General for a while. The awards were negotiated with the Association and management, and consolidated to provide coverage for all surveyors in various parts of the Department.

The SSA did successfully negotiate to have a representative on the 'Department Committee' enabling active interaction with management on industrial matters. The main issues during this period were award negotiations and the continued deployment of surveyors within the Department.

Following extensive discussions by members at the SSA Annual General Meetings from 2011 to 2014, questioning the relevance and effectiveness of this body, a motion was finally put at the 2014 AGM "that actions be initiated to dissolve the NSW Staff Surveyors' Association and advise members accordingly at the next APAS conference." The remaining assets are to be transferred to APAS on dissolution. This heralded the end of 90 years of proud service to its members and the Department of Lands. An almost complete list of all conferences and executive positions is collated in Table 1.

3 PROPOSAL TO FORM APAS

During the 1980s and early 1990s, employed surveyors of other government organisations and authorities were continually asking to join the Staff Surveyors' Association. This was mainly because their numbers were small and there was no mechanism for them to have a collective voice. The only broad organisation that existed at that time was a loose coalition that held an annual Government Surveyors Dinner that was instigated by L.N. Fletcher.

The broader view of membership to the Association was discussed when surveyors of the Forestry Commission joined SSA. Then the surveyors of the Maritime Services Board approached the Association to join (SSA Conf. 1977). The issue came to a head in November 1981 when the Department of Lands and the Department of Local Government amalgamated. Do surveyors employed in other organisations, albeit amalgamated with Lands, qualify as staff surveyor members? This seems to be the genesis of the formation of a combined government employed surveyor organisation.

In order to address this issue, discussions began within the Association to form a Government Surveyors' Association. At the April 1993 Conference held in Gerringong, a motion was passed to "sponsor a Government Surveyors' Association, by preparing a draft constitution." It was also moved that the next conference be promoted by the Association as a "Government Surveyors' Conference" (SSA Conf. 1993). At the SSA Executive meeting in September 1993, the President L.S. Gardner tabled a proposed set of rules to merge with the Government Surveyors' Association. At the AGM of the SSA Conference it was moved "that the SSA make provision for up to $1,500 seed funding to be provided for the creation of the Public Sector Surveyors' Association." It was noted in the minutes after this motion was passed that "this motion was one of the Association's most momentous decisions, and it should be proud of the initiative it has taken in this matter" (SSA Conf. AGM 1994).

Table 1: Staff Surveyors' Association – Executive and conference information.

Year	Conf. No.	No. Members (attending Conf)	Theme Title	Venue	President	Vice-Presidents	Hon. Secretary & Treasurer	Hon. Secretary	Councillors
1920	Inaugural	56			CJ Harnett		A Dunn		
1921	1	68		Royal Society's Office Sydney	FP Brown	CJ Harnett & SA Giraud	EH Dunlop		EA MacKenzie, HB Corlis, PR Drummond, JJ Baker, FJ Bootle, JS Turner, D Herborn, TW Watson
1922	2	100			FP Brown	CJ Harnett & HG Barrie	EH Dunlop		SA Giraud, JJ Baker, FJ Bootle, HB Corlis, PR Drummond, FC Della Ca, JS Turner, RM Drummond,
1923	3	130			HG Barrie	JJ Baker & HB Corlis	EH Dunlop		PR Drummond, CJ Harnett, LS Ferrier, D Herborn, RM Drummond, FP Brown, A Max Allen, JS Turner
1924	4				HG Barrie	LS Ferrier & HB Corlis	MJ Cronin		EH Dunlop, JS Turner, JJ Baker, WJ Close, PR Drummond, RM Drummond, EA MacKenzie, TW Watson
1925	5				JJ Baker	EH Dunlop & LS Ferrier	MJ Cronin		JJ Baker, HB Corlis, MJ Cronin, EH Dunlop, FC Della Ca, LS Ferrier, SA Giraud, EA MacKenzie, DS Mulley, GL Sheaffe, AH Stephen, TW Watson
1926	6				HB Corlis	JS Turner & DS Mulley	FC Della Ca		HB Corlis, AG Close, MJ Cronin, FC Della Ca, LS Ferrier, H Godfrey, RF Massie, DS Mulley, LA Proust, GL Sheaffe, AH Stephen, JS Turner
1927	7	57			JS Turner	DS Mulley & AG Close	FC Della Ca		AG Close, MJ Cronin, FC Della Ca, LS Ferrier, H Godfrey, RF Massie, DS Mulley, GL Sheaffe, AH Stephen, JS Turner, TW Waldron, CT Webb
1928	8				DS Mulley	AG Close & FH Nowlan	FC Della Ca		AG Close, MJ Cronin, FC Della Ca, HG Evans, H Godfrey, LA Kimber, RF Massie, DS Mulley, FH Nowlan, AH Stephen, TW Waldron, CT Webb
1929	9				DS Mulley	FC Della Ca & FH Nowlan	RF Massie		A Max Allen, MJ Cronin, FC Della Ca, HG Evans, H Godfrey, LA Kimber, RF Massie, DS Mulley, FH Nowlan, AH Stephen, JS Turner, CT Webb
1930	10				DS Mulley	FC Della Ca & CT Webb	RF Massie		A Max Allen, MJ Cronin, FC Della Ca, HG Evans, H Godfrey, LA Kimber, RF Massie, DS Mulley, FH Nowlan, AH Stephen, JS Turner, CT Webb
1931			Cancelled						
1932	11				DS Mulley	FC Della Ca & CT Webb	RF Massie		A Max Allen, FC Della Ca, CE Elphinstone, HG Evans, AC Fairley, H Godfrey, LA Kimber, RF Massie, DS Mulley, AHH Stephen, H Sheppard, CT Webb
1933			Cancelled						
1934	12				FC Della Ca	HG Evans & AC Fairley	CE Elphinstone		A Max Allen, FC Della Ca, CE Elphinstone, HG Evans, AC Fairley, LA Kimber, RF Massie, DS Mulley, AC Robb, AHH Stephen, H Sheppard, CT Webb
1935	13				H Godfrey	CT Webb & AC Fairley	CE Elphinstone		A Max Allen, FC Della Ca, CE Elphinstone, H Godfrey, RF Massie, DS Mulley, AC Robb, GL Sheaffe, H Sheppard, RB Stokes, CT Webb
1936	14				H Godfrey	CE Elphinstone & AP Lipscomb	AA Peirce		A Max Allen, HEH Barr, FC Della Ca, CE Elphinstone, AC Fairley, H Godfrey, AP Lipscomb, R Mostyn, AA Peirce, RB Stokes, GW Vincent, CT Webb
1937	15				FC Della Ca	CE Elphinstone & HJ Stone	AA Peirce		HEH Barr, FC Della Ca, CE Elphinstone, H Godfrey, LA Kimber, DS Mulley, AA Peirce, AC Robb, RB Stokes, GL Sheaffe, HJ Stone, GW Vincent

Year	Conf. No.	No. Members (attending Conf)	Theme Title	Venue	President	Vice-Presidents	Hon. Secretary & Treasurer	Hon. Secretary	Councillors
1938	16				FC Carr	CE Elphinstone & HJ Stone	AA Peirce		HEH Barr, FC Carr, CE Elphinstone, AS Hutchinson, EA Johnson, RF Massie, AA Peirce, AC Robb, RB Stokes, HJ Stone, GW Vincent, CT Webb
1939	17				FC Carr	CE Elphinstone & HJ Stone	GW Vincent		HEH Barr, FC Carr, CE Elphinstone, H Godfrey, GF Hendy-Pooley, AS Hutchinson, R Mostyn, AA Peirce, AC Robb, RB Stokes, HJ Stone, GW Vincent
1940	18	45			FC Carr	CE Elphinstone & HJ Stone	GW Vincent		HEH Barr, FC Carr, CE Elphinstone, H Godfrey, GF Hendy-Pooley, AS Hutchinson, R Mostyn, AA Peirce, AC Robb, RB Stokes, HJ Stone, GW Vincent
1941	19	44	*7 on war duties*		GW Vincent	H Godfrey & HJ Stone	AS Hutchinson		HEH Barr, FC Carr, CE Elphinstone, H Godfrey, GF Hendy-Pooley, AS Hutchinson, R Mostyn, AA Peirce, AC Robb, RB Stokes, HJ Stone, GW Vincent
1942			*Cancelled due to war*						
1943			*Cancelled due to war*						
1944	20	41			GW Vincent	GL Sheaffe & AA Peirce	AS Hutchinson		HEH Barr, FC Carr, CE Elphinstone, AS Hutchinson, R Mostyn, RF Massie, R Mostyn, AA Peirce, JN Prior, AC Robb, GL Sheaffe, RB Stokes, GW Vincent
1945	21	38			GW Vincent	AA Peirce & FL Mathews	HEH Barr		HEH Barr, FC Carr, CE Elphinstone, AS Hutchinson, FL Mathews, VF O'Donohue, AA Peirce, JN Prior, PE Raymond, GL Sheaffe, RB Stokes, GW Vincent
1946	22	44			GW Vincent	CE Elphinstone & AC Robb	HEH Barr		HEH Barr, FC Carr, CE Elphinstone, AS Hutchinson, AC Robb, J Middleton, LN Fletcher, JN Prior, PE Raymond, GW Anderson, RB Stokes, GW Vincent
1947	23	43			GW Vincent	CE Elphinstone & AC Robb	HEH Barr		GW Anderson, HEH Barr, FC Carr, CE Elphinstone, LN Fletcher, J Middleton, AS Hutchinson, JN Prior, PE Raymond, AC Robb, RB Stokes, GW Vincent
1948	24	35			CE Elphinstone	AS Hutchinson & RB Stokes	CC Bradley		GW Anderson, HEH Barr, PJ Firth, LN Fletcher, TP Keig, T Allsop, JN Prior, AS Wolstenholme
1949	25	37			HEH Barr	AS Hutchinson & RB Stokes	TP Keig		GW Anderson, CC Bradley, AS Wolstenholme, JN Prior, FF Kell, LN Fletcher, CW Prince, AW Bonner
1950	26	32			HEH Barr	AS Hutchinson & RB Stokes	TP Keig		GW Anderson, CC Bradley, AS Wolstenholme, FF Kell, CW Prince, JN Prior, AW Bonner, LN Fletcher
1951	27								
1952	28								
1953	29								
1954	30								
1955	31								
1956	32	46			PJ Firth	LN Fletcher & CC Bradley	JB Martin		CW Prince, RV Clarke, CR Champion, JN Fenton, LH Anderson, B Stapleton, JW Power, EF Higgs
1957	33	42							
1958	34								

Year	Conf. No.	No. Members (attending Conf)	Theme Title	Venue	President	Vice-Presidents	Hon. Secretary & Treasurer	Hon. Secretary	Councillors
1959	35	47			LN Fletcher	CW Prince & CC Bradley	JB Martin		JC Paterson, D O'Keefe, JN Fenton, JW Power, CRB Simpson, J Darby, B Stapleton, KJ Fallon
1960	36			Maritime Services Board Hall, Sydney	LN Fletcher				
1961	37	45		Maritime Services Board Hall, Sydney	CW Prince	JW Power & JN Fenton	JB Martin		D O'Keefe, DA Johnstone, B Stapleton, CR Champion, RV Clark, K Hopkins, JB McLean, J Darby
1962	38	Members 48 Associates 18		Institution of Surveyors NSW, Sydney	CW Prince	JW Power & JN Fenton	JB Martin		D O'Keefe, DA Johnstone, B Stapleton, CR Champion, RV Clark, K Hopkins, J Darby, JC Paterson
1963	39	Members 50 Associates 36		Maritime Services Board Hal, Sydney	JW Power	D O'Keefe & DA Johnstone	JB Martin		J Darby, WA Searl, CR Champion, J Adams, F Higgs, JC Paterson B Stapleton, K Hopkins
1964	40	Members 48 Associates 37		Maritime Services Board Hall, Sydney	D O'Keefe	J Darby & J Adams	JB Martin		CR Champion, B Stapleton, JR Roberts, K Hopkins, JC Paterson, CRB Simpson, AET Jackson, WA Searl
1965	41	Members 59 Associates 18		Maritime Services Board Hall, Sydney	J Darby	JC Paterson & CR Champion	JB Martin		WA Searl, J Adams, AJ Robinson, K Hopkins, B Stapleton, P Seidel, WP Kelly, JB McLean
1966	42	Members 58 Associates 28		Maritime Services Board Hall, Sydney	JC Paterson	CR Champion & K Hopkins	WG Spink		JB McLean, WA Searl, EP McAnespie, WP Kelly, P Seidel, J Adams, B Stapleton, AET Jackson
1967	43	Members 66 Associates 47 Total = 113		Maritime Services Board Hall, Sydney	JC Paterson	K Hopkins & WG Spink	JB McLean		IH Marshall, CR Champion, JDK Walker, AET Jackson, J Adams, EP McAnespie, JD Osmond, WP Kelly
1968	44	94		Maritime Services Board hall, Sydney	K Hopkins	WG Spink & B Stapleton		WH Burrell	JB Martin, CR Champion, JDK Walker, AET Jackson, GJ Newham, EP McAnespie, JD Osmond, WP Kelly
1969	45	108		Maritime Services Board Hall, Sydney	WG Spink	WP Kelly & CR Champion		WN Thomson	B Stapleton, JW Osmond, JDK Walker, JB Martin, GJ Newham, EP McAnespie, AET Jackson, WH Burrell
1970	46	99 plus 14 trainees		Maritime Services Board hall, Sydney	WG Spink	CR Champion & WP Kelly		JB Frost	B Stapleton, GJ Newham, JDK Walker, JB Martin, HG Helsham, EP McAnespie, AET Jackson, JR Wallace
1971	47			Maritime Services Board Hall, Sydney	WG Spink	CR Champion & AET Jackson		JB Frost	AC Leard, GJ Newham, JDK Walker, JB Martin, HG Helsham, PG Knights, AE Smythe, JR Wallace
1972	48	Members 72 Associates 30 Students 27 Total = 129		Maritime Services Board hall, Sydney	AET Jackson	WG Spink & JDK Walker		JB Frost	AC Leard, JR Wallace, JW Osmond, JB Martin, DAK Vincent, GJ Newham, HG Helsham, PG Knights
1973	49	Members 75 Associates 38 Students 1 Total = 114		Maritime Services Board Hall, Sydney	AET Jackson	WG Spink & JR Wallace		BG Preston	AC Leard, RTJ Benjamin, JW Osmond, JB Frost, HG Helsham, PG Knights, JT Judge, LB Scrivener
1974	50	Members 65 Associates 17 Students 21 Total = 115		Maritime Services Board Hall, Sydney	AET Jackson	JR Wallace & JB Frost		LB Scrivener	JR Read, RTJ Benjamin, JW Osmond, PE Naylor, HG Helsham, BG Preston, RJ Mullins, GW Middleton
1975	51	Members 66 Associates 40 Students 1 Total = 107		State Office Theatrette, Sydney	JB Frost	TC Ryan & R Beeston		HG Helsham	LB Scrivener, RTJ Benjamin, PE Naylor, JR Read, RJ Mullins, R Beeston, KJ Graf, JT Judge
1976	52	no record found		Teachers' Federation Auditorium, Sydney	JB Frost	AC Leard & JDK Walker		HG Helsham	LB Scrivener, RTJ Benjamin, PE Naylor, TC Ryan, RJ Mullins, CW Starkey, KJ Graf, JT Judge

Year	Conf. No.	No. Members (attending Conf)	Theme Title	Venue	President	Vice-Presidents	Hon. Secretary & Treasurer	Hon. Secretary	Councillors
1977	53	no record found		Institute of Technology, Sydney	JDK Walker	TC Ryan & BG Preston		HG Helsham	GJ Kelly, JR Read, RTJ Benjamin, JT Judge, PE Naylor, CE Neale, GW Middleton, JB Frost
1978	54	no record found		Mitchell College of Advanced Education, Bathurst, NSW	JDK Walker	HG Helsham & JT Judge		BG Preston	PE Naylor, JR Read, RTJ Benjamin, TC Ryan, I Robins, G Kelly, RJ Mullins, J Fletcher
1979	55	Members 85 Associates 38 Graduates 9 Total = 132		Macquarie University, North Ryde NSW	BG Preston	JT Judge & JD Walker		RD Lister	PE Naylor, TC Daly, JR Read, CE Neale, I Robins, G Kelly, RJ Mullins, R Cubis
1980	56	Members 82 Associates 34 Graduates 4 Total = 120		University of Newcastle, Shortland NSW	BG Preston	TC Ryan & I Robins		P Barclay	RJ Mullins, J Judge, G Dodd, PE Naylor, K Graf, D Kinlyside, W Watkins, CE Neale
1981	57		Development of Environment	Sydney NSW					
1982	58	114	Surveying Mapping, Training, Safety, Change	Bathurst NSW	J Read				
1983	59	120	Land Management: The Expanding Horizon	Carrington Hotel, Katoomba NSW	HG Helsham	P Naylor & CE Neale		G Samuel	R Cole, D Kinlyside, J Robertson, RC Lander, W Watkins, I Robins, A Woodruff, G Baitch
1984	60		Footsteps in Time	Sydney NSW					
1985	61		Surveying our Natural Resources	Nowra NSW	CE Neale				
1986	62	not reported	Surveying New Fields	Salamader Hotel, Soldiers Point NSW	G Baitch	J Judge & TC Daly		RC Lander	J Filocamo, G Dickson, S McDonald, P O'Kane, R Mullins, G Flood, A Woodruff, T Ferrier
1987	63								
1988	64	not reported		Artarmon Inn, Artarmon NSW	TC Daly	D Halls & J Filocamo		J Robertson	T McInnes, G Norrie, G Stewart, G Songberg, T Ferrier, B Hanna, T Duffy, B McCann
1989	65		The Next 200 Years: Lessons from the Past, Direction for the Future	Nowra NSW					
1990	66								
1991	67		Land Information / Land Management: Working Together	Bathurst NSW					
1992	68		Conservation in the 90s	Penrith NSW					
1993	69			Gerringong NSW					
1994	70		Surveying Geomatics	Goulburn Police Academy, Goulburn NSW					
1995	71		The Challenge	Port Macquarie NSW	L Gardner	A McAnespie		G Baitch	R Ellis, R Birse, R Mullins, G Dickson
1996	72	34	Surveying our Resources	Leura NSW	G Stewart	K Thompson		G Baitch	R Ellis, G Groves, B Hurcum
1997	73	44	Capitalising on Diversity	Eagle Hawk Hill NSW	K Thompson	G Groves		L Gardner	O Moss, K Green, R Birse
1998	74	55	Focus on 2000	Country Comfort Motel, Salamander Bay NSW	K Thompson	G Groves		L Gardner	O Moss, K Green, R Birse
1999	75		Look now Cook!	Murramarang Resort, South Durras NSW	K Thompson	R Ellis		L Gardner	P O'Kane, G Songberg, G Stewart
2000	76	25	Surveying Diversity	Pacific Bay Resort, Coffs Harbour NSW	R Ellis	P Ragen		L Gardner	T Daly, G Stewart, G Songberg

Year	Conf. No.	No. Members (attending Conf)	Theme Title	Venue	President	Vice-Presidents	Hon. Secretary & Treasurer	Hon. Secretary	Councillors
2001	77	16	Tending the Vine	Hunter Valley Gardens, Pokolbin NSW	R Ellis	P Ragen		L Gardner	T Daly, G Stewart, G Songberg
2002	78	14	One Small Step for Surveying, One Giant Leap for Spatial Sciences	Eagle Hawk Conference Centre, Sutton NSW	R Ellis	P Ragen		L Gardner	T Daly, G Stewart, G Songberg
2003	79	15	Surveying by the Sea	Novatel Northbeach, Wollongong NSW	P Ragen	G Stewart		L Gardner	T Daly, P O'Kane, G Songberg
2004	80	14	The Tides of Change	Citigate Convention Centre, The Entrance NSW	P Ragen	G Stewart		L Gardner	T Daly, P O'Kane, G Songberg
2005	81	18	Bridging the Information Gap	Coach House Marina, Batemans Bay NSW	P Ragen	G Stewart		L Gardner	T Daly, P O'Kane, G Songberg
2006	82	14	Shorelines to Borders	White Sands Conference Centre, Shoal Bay, Port Stephens NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2007	83	18	To the Territorial Limits	Hotel Heritage, Narrabundah ACT	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2008	84		Space and Time	Mantra Ettalong Beach, Ettalong NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2009	85	18	Backsight to the Future	Mantra Ettalong Beach, Ettalong NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2010		19	*AGM only – no conference due to FIG2010 in Sydney*	Shelbourne Hotel, Sydney NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2011	86	17	Challenging Our Horizons	Citigate Conrod Straight, Bathurst NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2012	87	15	From Port t Portals: Surveying and Spatial Directions	Novotel North Beach, Wollongong NSW	P Ragen	G Stewart		L Gardner	D Kennedy, P O'Kane, G Songberg
2013	88	12	Capitalising on Our Position	Canberra Rex Hotel, Canberra ACT	P Ragen	W Fenwick		L Gardner	D Kennedy, K Thompson, G Songberg
2014	89	12	Grape Expectations: Pressing Forward	Mercure Resort Hunter Valley Gardens, Pokolbin NSW	P Ragen	W Fenwick		L Gardner	D Kennedy, K Thompson, G Songberg
2015	90		Banana Jubilee: 20 Fruitful Years	Novotel Pacific Bay Resort, Coffs Harbour NSW	N/A	N/A	N/A	N/A	N/A

The inaugural Association of Public Authority Surveyors Conference was held in April 1994 at the Goulburn Police Academy, incorporating the 70th SSA Conference. The new rules of APAS endorsed at this conference allowed members from all levels of government (being state, federal and local) to join this body. The new body would provide an annual conference and enable members to make representations to various other institutions. However, APAS would have no industrial purpose and not be affiliated with any other industrial body. APAS and SSA ran in parallel for the next 20 years, until the dissolution of SSA in 2015.

4 CONFERENCE PROCEEDINGS, PAPERS AND MEMBERSHIP RECORDS

Since the first SSA Conference in 1921 to the commencement of APAS in 1994, the Association kept assiduous records of the annual meetings. Originals are held in a steel cabinet the Survey section of LPI in Bathurst. It is hoped that one day these may be converted into electronic records. The Proceedings of each conference are a record of what was said by delegates and record important (and sometimes unimportant) decisions made by members and guests. One of the early issues that emerged from the reading of the proceedings is that the Surveyor General and Under Secretary of the Department seemed to have an influence on the content of the record of proceedings. In addition, it often took longer than a year to finalise the proceedings.

Conference papers that were presented, were either summarised in the Proceedings, or published separately (and initially individually). Records of the papers are not comprehensive, nor organised in any fashion, though most are located in the abovementioned cabinet. Ken Green, a retired past member of the Association, created a spreadsheet of all the titles of papers presented to the Association prior to the mid 1970s.

In an effort to identify the useful papers and determine whether or not any are suitable for re-publication, the SSA ‘Papers and Publications Committee’ investigated all past papers. The Committee undertook this task initially in 1967 of papers presented from 1923 to 1950. This Committee was re-formed in 1969 to assess papers from 1951 to 1969. Past papers were categorised as follows:

- N – no use
- C – could be useful (generally indicated that reader was not conversant with subject)
- B – provides background to recent subject
- U – useful
- G – general paper
- P – new paper
- H – historical

This meant that each member of the committee read each and every paper. The committee members were J. Judge (convenor), D.A.K. Vincent, J. Wallace, R. Mullins, J.B. Martin and R.B. Stokes. The results of this review are shown in Table 2.

Table 2: Staff Surveyors' Association – Comments on papers from 1921 to 1968.

Year	Paper	Vincent	Judge	Wallace	Mullins	Martin	Stokes
1921	1	C		N	U/H		
	2	C		N	B		
1922	1	B		N	B		
	2	B		N	N		
	3	B		N	B		
	4	C		C	C		
	5	B		N	U		
1923	1	N	N	N	B		
	2	C	C	N	U		
	3	C	C	N	U		
	4	N	N	N	U		
	5	N	N	N	N		
1924	1	H		H	H		
	2	N		N	N		
	3	N		N	N		
1925	1	C	C	N	H		
	2	N	C	N	U		
	3	N	C	N	N		
	4	C	C	N	H		
1926	1	B	C	C			
	2	CG	CG	H			
1927	1	C	C	C			
	2	C	C	C			
1928	1	C	C	C			
	2	C	C	C			
	3	C	C	C			
	4	C	C	C			
	5	C	C	C			
	6	C	C	C			
	7	C	C	C			
	8	C	C	C			
	9	C	C	C			
	10	C	C	C			
	11	C	C	C			
	12	C	C	C			
	13	C	C	C			
1929	1	C	C	C			
	2	C	C	C			
	3	U	U	U			
	4	C	C	C			
	5	H	H	H			
	6	N	N	N			
	7	N	N	N			
1930	1	U	U	U			
	2	N	N	H			
	3	U	C	U			
	4	C	N	N			
	5	C	C	C			
	6	C	N	N			
1932	1	B	CP		N		
	2	B	C		U		
	3	B	C		B		
	4	B	C		BH		
1934	1	H	B	GH	B		H
	2	N	C	N	B		N
	3	N	GP	N	H		
1935	1	H	N	N	H		H
	2	C	C	C	B		N
	3	H	GP	N	H		H
1936	1	N	N	N	B		N
	2	B	C	N	B		N
	3	N	C	N	B		N
	4	N	N	N	N		N
1937	1			N	B		
	2			N	N		
	3			U	U		
1938	1	GH	G	GH			
	2	N	C	N			
	3	N	N	N			
1939	1	U	N	U			
	2	N	CP	N			
1940	1	H	UG	H			
1941	1	C	C				
	2	H	UG				
1944	1	N	C				
1945	1	N	B	N			
	2	N	N	N			
1946	1	H	B	H			
	2	N	C	N			
	3	H	B	H			
1947	1	H	C	H			
	2	C	C	C			
1948	1						
	2	HU	N				
	3	C	C				
	4	N					
1949	1	U	C				
	2	C	C				
	3						
1950	1						
	2	N	CB				
	3	C	U				
1951	1		C	N	B	B	NP
1952	1		G	U	U	C	U
1953	1		H	H	H	H	HP
1954	1						
	2						
	3						
	4						
1955	1						
	2		C	P	P	C	
	3						
	4						
	5		GC	U	UP	P	U
1956	1						
	2						
	3						
	4						
	5	U	U	U	B	BP	NP
	6	G	G	C	U	U	U
	7	U	U	C	U	BP	U
	2.T	C	U	C	C	C	
	2.C	C	U	C	C	C	
	2.M	C	U	C	C	C	
1957	1	C	C	C	U	U	
1958	1						
	2	H	U	N	H	H	
	3	Good	Good	U	U	U	
	4	Good	Good	Good	U	G	
	5						
	6						
	7						
	8						
1959	1	BP	U		P	BP	P
	2	Good	Good		U	Good	U
	3						
	4	H	HU		H	H	H
	5	C	U		P	C	P
	6	C	U		B	C	U
	7	N	N		N	N	N
	8	N	N		N	N	N
	9	?	?			?	
1960	1	BP	HG		B	GH	H
	2	C	U		U	U	U
	3						
	4						
	5	U	U		U	U	U
	6						

Year	Paper	Vincent	Judge	Wallace	Mullins	Martin	Stokes
1960	7	G	G		U	G	U
	8	U	Good		U	Good	U
	9						
	10	UP	Good		U	Good	U
1961	1		N	N	N	N	N
	2	Good	Good	U	U	Good	U
	3	Good	Good	U	U	Good	U
	4	Good	Good	U	U	Good	U
	5		N	N	N	N	N
	6	C	C	C	U	C	U
	7	C	C	C	C	C	UP
	8		N	N	N	N	N
	9	C	U	C	B	C	?
1962	1	U	Good	U	U	U	U
	2	P	Good	UP	P	P	U
	3	P	Good	UP	P	P	P
	4	C	C	P	P	C	P
	5	U	Good	U	U	U	U
	6	U	Good	U	U	U	U
	7	N	N	N	N	N	N
	8	N	N	N	U	C	N
	9	U	Good	U	U	U	U
	10	U	U	C	U	U	U
1963	1	U	Good	U	U	U	U
	2	N	N	N	N	N	N
	3	U	Good	U	U	U	U
	4	B	C	B	B	B	U
	5	H	N	H	H	N	N
	6	P	Good	B	B	B	UP
	7	U	Good	U	U	U	U
	8	U	Good	U	U	U	U
	9	N	N	N	N	N	N
1964	1	P	Good	P	P	P	P
	2	N	N	N	N	C	N
	3						
	4	U	Good	U	U	U	U
	5	P	Good	UP	P	CP	P
	6	N	N	N	N	N	N
	7	N	N	N	N	N	N
	8	N	N	N	N	N	N
	9	N	N	N	N	N	N
	10	C	C	C	N	C	N
	11	Good	Good	U	U	U	U
	12	U	Good	U	H	U	U
	13	N	N	N	N	N	N
1965	1		N				
	2	N	C	U	U	U	U
	3	N	N	N	N	N	N
	4	N	N	N	N	N	N
	5	N	N	N	N	N	N
	6	U	C	U	U	U	U
	7	Good	Good	Good	Beauty	U	U
	8	N	N	N	N	N	N
	9	N	N	N	N	N	N
1966	1	Good	Good	U	Good	U	U
	2		N		N		
	3	U	C	U	U	U	U
	4	N	N	N	N	N	N
	5	N	N	N	N	N	N
	6	H	Good	H	H	H	H
	7	U	C	U	U	U	U
	8	N	N	N	N	N	N
	9	N		N	N	N	N
	10	U	U	U	U	U	U
	11	U	Good	U	U	U	
	12	N	N	N	N	N	H
	13	N	C	N	N	U	N
	14	N		N		N	N
	15	N	N	N	N	N	N
1967	1	UH	Good	UH	H	UH	UH
	2						
	3	U	Good	U	U	U	U
	4	N	N	N	N	N	N
	5	Good	Good	U	U	U	U
	6	N	N	N	N	N	N
	7	N	N	N	N	N	N
	8	N	N	N	N	N	N
	9	N	U	U	N	U	
	10	N	N	N	N	N	N
	11	U	Good	U	U	U	U
	12	Good	Good	Good	U	U	U
	13	Good	Good	Good	U	U	U
	14	U	Good	U	U	U	U
	15	N	N	N	N	N	N
	16	N	N	N	N	N	N
1968	1						N
	2						N
	3						N
	4	U	C	U			U
	5	U		U			U
	6						
	7						
	8						
	9	N	N	N			N
	10						
	11	N	N	N			N
	12	N	N	N			N
	13						
	14						
	15	N	N	N			U
	16						

From the mid-1970s, the Association published a separate bound volume of papers presented at each conference. Since 1994, APAS has produced a bound volume of papers presented at Annual Conferences. SSA has kept good records of meetings in electronic form. From the 20th APAS conference in 2015 onwards, Proceedings are exclusively published online on the APAS website, which was launched in 2014 (APAS, 2015).

A full list of members' names was initially recorded in the early Proceedings. Then this practice lapsed. Full membership lists reappeared in the proceedings in the 1970s. In 1972, the membership details were entered into a Department of Lands computer and published in full in the Proceedings. Membership records are now managed at the LPI office in Bathurst.

5 TEAM BUILDING: SSA AND GOLF

The first record of a golf game associated with the Annual Conference is that on 16 April 1936: "a team of golfers from the Association spent 'a half day field' playing against a team from Head Office, the venue being the Manly Golf Links." The field officers lost (SSA Conf. 1936, p.37). The second golf game recorded that "during the afternoon of 1 April 1937, played at Manly Golf Links" (SSA Conf. 1937, p.17). The third golf game recorded was played during the afternoon of 21 April 1938. Members of the Association played against Head Office for the silver cup presented by Messrs Allman and Allen called the 'Allman Allen - Golf Cup'. Mr G.F. Allman was the Under Secretary for Lands and Mr A.M. (Max) Allen was the Surveyor General at the time. The venue for that game was the Moore Park Golf Links (SSA Conf. 1938, p.17). This trophy was contested regularly until 1977, then suddenly stopped, and was never contested again.

The 'Sinclair Cup' is contested for the best scratch result from an Association member. It is assumed that this cup was named after The Hon. A. Sinclair, who was the Minister for Lands at that time (SSA Conf. 1941, p.17). The Sinclair Cup is a handsome cup made of silver and now a genuine antique of significant historical value. This trophy has been contested every year, except during the war years, to this day.

The '1971 Anniversary Trophy - Handicap Event' has been contested every year since 1971. Unfortunately, this silver plated trophy has not weathered the years well and is now quite tarnished. A list of winners' names is shown in Table 3.

6 AWARDS

The 'Jack Frost Award' was created in memory of Jack Frost, a past President of the Association who died of skin cancer in the 1980s. This award was made in recognition of his contribution to the Association. It was awarded for the 'best paper' presented at the Annual Conference by an Association member. A sum of $1,500 was set aside to fund the awarding of a prize each year.

In 1996, it was agreed that the 'Jack Frost Award' be extended to include all APAS/SSA members for the best paper at the conference (SSA Conf. AGM 1996, p.2). In 2012, it was substituted by the 'Keith Haddon Memorial Award' following Keith's death. Keith Haddon was one of the Association's life members and the 'behind the scenes' worker, who generally managed all the logistics of every conference from 1986 till his retirement.

7 CONCLUDING REMARKS

The principal objective of this paper was to record the history of the Staff Surveyors' Association, now that it has ceased to exist. Over 90 years, the SSA has fulfilled a crucial role in the representation and professional development of Lands Department surveyors in particular and government surveyors in general. The SSA has spawned APAS, a broader association that will continue to perform this role into the future. This paper has shown how organisations that are alert to their environment can identify necessary changes, take bold initiatives and adapt to the times. The future is clearer with awareness of the past.

Table 3: Staff Surveyors' Association – Golf results.

Allman Allen - Golf Cup	
1936	SSA
1937	SSA
1938	Head Office
1939	Head Office
1940	– no play –
1941	– no play –
1942	– no play –
1943	– no play –
1944	– no play –
1945	– no play –
1946	– no play –
1947	– no play –
1948	SSA
1949	Head Office
1950	Head Office
1951	Head Office
1952	Head Office
1953	Head Office
1954	Head Office
1955	Head Office
1956	Head Office
1957	Head Office
1958	Head Office
1959	Head Office
1960	Head Office
1961	Head Office
1962	SSA
1963	SSA
1964	SSA
1965	Head Office
1966	SSA
1967	SSA
1968	SSA
1969	SSA
1970	Head Office
1971	SSA
1972	SSA
1973	Head Office
1974	Head Office
1975	SSA
1976	SSA
1977	Head Office
Sinclair Cup	
1938	RCC Hewitt
1939	EC Gleeson
1940	– no play –
1941	– no play –
1942	– no play –
1943	– no play –
1944	– no play –
1945	– no play –
1946	– no play –
1947	– no play –
1948	CEE Elphinstone
1949	J Davis
1950	AE Thomas
1951	J Davis
1952	R Fitzgerald
1953	R Fitzgerald
1954	R Fitzgerald
1955	A Wardley
1956	N Simmons
1957	J Darby
1958	N Simmons
1959	N Simmons
1960	N Simmons
1961	J Darby
1962	D Sheedy
1963	J Darby
1964	JD Walker
1965	J Darby
1966	JD Walker
1967	JD Walker
1968	JD Walker
1969	AE Jackson
1970	JD Walker
1971	JD Walker
1972	JD Walker
1973	W Mininch
1974	J Darby
1975	JD Walker
1976	JD Walker
1977	J Darby
1978	JD Walker
1979	JD Walker
1980	JD Walker
1981	JD Walker
1982	G Helsham
1983	JD Walker
1984	JD Walker
1985	G Helsham
1986	G Helsham
1987	G Helsham
1988	G Helsham
1989	G Helsham
1990	G Helsham
1991	G Helsham
1992	G Helsham
1993	K Haddon
1994	R Ellis
1995	O Moss
1996	O Moss
1997	O Moss
1998	K Haddon
1999	O Moss
2000	K Haddon
2001	K Haddon
2002	K Haddon
2003	K Haddon
2004	R Fish
2005	O Moss
2006	K Haddon
2007	K Haddon
2008	O Moss
2009	K Haddon
2010	– no play –
2011	O Moss
2012	D Kennedy
2013	O Moss
2014	G Baitch
2015	
1971 Anniversary Trophy	
1971	A Graham
1972	R Benjamin
1973	S Williams
1974	S Williams
1975	HP Jackson
1976	J Cuffe
1977	J Smith
1978	G W......
1979	R Cole
1980	P Pluis
1981	P Pluis
1982	P Pluis
1983	T Duffy
1984	G Helsham
1985	L Griffin
1986	G Dickson
1987	R Ellis
1988	J Miller
1989	G Dickson
1990	I Pengelly
1991	G Dickson
1992	R Mullins
1993	D Mills
1994	L Gardner
1995	G Jones
1996	K Haddon
1997	K Haddon
1998	J Eggleston
1999	R Cole
2000	R Ellis
2001	O Moss
2002	G Baitch
2003	L Gardner
2004	K Haddon
2005	G Jones
2006	K Thompson
2007	G Baitch
2008	G Jones
2009	G Baitch
2010	– no play –
2011	G Baitch
2012	R Ellis
2013	B Hurcum
2014	R Ellis
2015	

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Thanks go to Les Gardner of LPI, who in recent times has been the archivist and guardian of all the SSA records, which are securely housed in a steel cupboard in the Survey section of LPI in Bathurst. Thanks also go to Ken Green and John Read, who are active members of the Seniors' Group of the Institution of Surveyors NSW (and past members of SSA) and generously provided additional information for this paper.

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Locating the Flag Staff at Bathurst

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ABSTRACT

The Bathurst Flag Staff was erected on Friday, 5 May 1815. On Sunday, 7 May 1815, the British Union Jack was ceremoniously raised and the Town of Bathurst proclaimed. 200 years later, Bathurst Regional Council proposes to replicate the Flag Staff as an integral event in its bicentenary. But where was the Flag Staff? Because doubt existed, it was put to surveyors (who else?) to locate where it originally was. The geographical position officially stated in 1815 places the Flag Staff about 4 km east of the actual position. Although relatively precise for its time, this discrepancy led to a fascinating journey of historic and scientific discovery looking at the unchanging certainty of the celestial sphere and the uncertain determination of absolute position on the ground. This paper looks at the advances in the determination of the absolute geographical position of the Bathurst Flag Staff over the last 200 years, with regard also to the position of Sydney upon which the geographical position of Bathurst (and towns elsewhere in New South Wales) was initially based. The Flag Staff was also the official surveyed base point, reference mark or origin for the setting out of the proposed Town of Bathurst and for the exploration and survey of the Central and Western Districts of New South Wales. This paper looks back at original survey plans and retrospectively determines the position of the Flag Staff from these plans and the present occupations of the Town of Bathurst originally set out from the Flag Staff. It finds that an undated, unacknowledged and unsubstantiated rock cairn with a brass plaque purporting to be "the exact location of the Flag Staff" is not where it should be. This paper sets out the surveying methodology used to determine where it should be, with surprising precision.

KEYWORDS: *Bathurst, Flag Staff, location, history*.

1 INTRODUCTION

The Town of Bathurst was proclaimed by Governor Lachlan Macquarie at the Flag Staff on Sunday, 7 May 1815. However, the subsequent purpose of the Flag Staff was to provide a reference mark for aligning the town blocks of Bathurst and for a point of commencement for exploration of the inland of New South Wales (NSW). The Flag Staff was unique, and as a reference mark it predates the 1818 Macquarie Obelisk in Macquarie Place, Sydney, used as the origin for road distances in NSW.

The 2015 Bicentennial Bathurst Flag Staff is to be erected centrally on a concrete slab concourse over an existing rock cairn to commemorate the proclamation of Bathurst. The relocated, unsubstantiated and undated rock cairn and brass plaque purports to be the site of the 1815 Flag Staff. However, the actual re-determined Flag Staff site is about 5.4 m closer to town. This paper contends that this actual position should be acknowledged to commemorate the equally if not more important location and surveying context of the Flag Staff.

2 STATED POSITION IN 1815

The dispatch by J.T. Campbell, Governor Macquarie's Secretary, on "Macquarie's First Tour beyond the Blue Mountains 10 June 1815" quotes: "*The site designed for the town of Bathurst, by observation taken at a flag-staff, which was erected on the day of Bathurst receiving the name, is situated at lat. 33º24'30" south, and in long. 149º37'45" east of Greenwich, being also about 27½ miles north of Government House in Sydney, and 94½ west of it bearing west 20º30' north 83 geographic miles, or 95½ statute miles, the measured road distance from Sydney to Bathurst being 140 English miles*."

In this context, the following should be noted:

- Conversion factors: One geographic mile is 1' arc along the earth's equator and 1.8553 km. One nautical mile is 1' arc along a great circle of the earth and 1.852 km. (These were mainly used in sea navigation.) One statute mile, also known as English mile, is 1.609344 km. This is also 1,760 yards, 5,280 feet and 80 chains. As with chains and links (1/100 chain), statute/English miles were commonly used in land measurement.
- Conversion of the stated distances to metric: 27½ miles = 44.257 km, 94½ miles = 152.083 km, 83 geographic miles = 153.99 km, 95½ statute miles = 153.692 km, 140 English miles = 225.3 km.

It is not known why it was necessary to cite geographic, rectangular and polar coordinates as they are mathematically inconsistent and imprecise (± ¼ mile or 400 m, see section 5), but it does tend to emphasise the location and position context of the 1815 Bathurst Flag Staff.

3 SURVEYING METHODS AND INSTRUMENTS

3.1 General

Whitehead (2003, 2004) describes Surveyor General John Oxley's 1817 and 1818 expeditions, providing an excellent account of the surveying methods and instruments of the time. The surveying instruments and methods used differed according to their purpose.

3.2 Land Surveying and Dead Reckoning

Compass or circumferentor and Gunther's chain were used in land surveying where the primary task was to dimension grants of portions and allotments as located on the ground. The natural boundaries, pegged corners and blazed lines formed the boundaries and these prevailed over the dimensions that described them. Bearings were referenced to Magnetic North and distances were in chains. Origins and azimuth were connections to existing surveys and the Bathurst Flag Staff was established for this purpose.

Dead reckoning uses land surveying methodology to determine absolute or geographical position for exploration and mapping. A compass/circumferentor and chain traverse was converted to geographical position by the following steps:

1. Summed into latitudes (N+, S-) and departures (E+, W-).
2. The bearing and distance were determined.
3. This was then adjusted for magnetic deviation to True North.
4. The Eastings and Northings were determined.

5. These were then converted to arc (i.e., in this instance, one chain = 1.53” of latitude and 1.284” of longitude).
6. Finally, they were added to known geographic coordinates to determine the new position sought.

3.3 Astronomical Observations

A second method used was astronomy, which had its origins in navigation practice where astronomical observations are taken using (then) a sextant, nautical tables and complex mathematics to determine True North, compass deviation from north, and mean solar time, latitude and longitude relative to Greenwich.

The observation used were:

- Amplitude observations (mean between sunrise and sunset).
- Double altitude observations (hour before and after noon).
- Altitude observations of the sun at noon.
- Lunar distances (angles between the moon and the sun or any of nine recorded stars). A Vernier sextant of the time could be read to 0.1’ with an experienced observer having a precision of ¼’ or 15” being about 400 m in latitude. Longitude by lunar distances could be anything up to 23 km out.

3.4 Astronomical Observation and Chronometer

The third method used a chronometer. This was used to determine the difference in time between local mean time at the point of observation and Greenwich. It much depended upon the reliability and treatment of the chronometer and its calibration, in this instance, the longitude of the time signal. A difference of 1” in time is 15” of arc or about 400 m.

4 THE LATITUDE AND LONGITUDE OF SYDNEY

The story behind the determination of the latitude and longitude of Sydney is one of patronage, unreliability, officialdom, and the improvement of precision over time. When considering the geographical position of the Bathurst Flag Staff, if Sydney’s position was imprecise, Bathurst was likely to be also.

4.1 The First Fleet and Lieutenant William Dawes

When the First Fleet arrived in Sydney, one of its first tasks was to determine its latitude and longitude as a reference point for future navigation. The first observatory was set up in July 1788 at Dawes Point (near the southern end of the Sydney Harbour Bridge). Lieutenant William Dawes of the Royal Marines, a competent astronomical observer, undertook the observations. He used instruments provided by the Board of Longitude and the Astronomer Royal, Rev. Nevil Maskelyne, including an astronomical quadrant, two telescopes, a Hadley’s sextant by Ramsden, two thermometers and a barometer. Also, importantly, he used a nautical almanac (first published in 1767), an astronomical clock by John Sheldon, and the Kendall No. 1 Marine Timekeeper (K1) made famous by Captain James Cook on his last two voyages.

Even though there were special instructions that the K1 chronometer had to be wound at noon every day, this was forgotten when it was transferred from the *Sirius* to the *Supply* at Cape

Town. It had run down and had to be reset by Dawes. When the First Fleet arrived at Sydney, Dawes had to determine the position of Sydney relative to Greenwich time by (1) Chronometer and by (2) the lunar distance method, a complex method championed by the Astronomer Royal.

4.2 Observations for Longitude from Dawes Point

The following observations were made (Figure 1):

- 1788 Dawes determination (1): The observatory at Dawes Point was then determined at latitude 33º52’30” S and longitude 151º19’30” E (being 10h 05m 18s east of Greenwich). This is 0º06’55” east of the actual location, being about 11 km to the east (and 4 km out to sea off Dover Heights). The magnetic declination was determined at 7º54’ E.
- 1788 Captain John Hunter’s determination (2): Coincidently, also in 1788, Captain John Hunter observed Dawes Point at latitude 33º51’50” S and longitude 151º13’ E, just east of the present day Government House.
- 1803 Matthew Flinders’ determination (3): Flinders observed Dawes Point at latitude 33º51’45.8” S, longitude 151º11’49.5” E (positioned in Darling Harbour). The magnetic declination was found to be 8º51’ E.

Hunter’s and Flinders’ observations are significantly close to the actual position, but who was to say whose longitude was right? Dawes was a protégé of the Astronomer Royal, had instruments and tables provided by him and the Board of Longitude, and, importantly, had the apparent endorsement of both, so his position determination was adopted. (This, it seems, continued to be adopted for many years.)

This is especially relevant as chronometers and Sydney clocks were then set to the time ball and noon gun (and later, the one o’clock gun) at the Dawes Point Battery, based (now seen in hindsight) on Dawes relatively incorrect determination. Also, for convenience, and being the official determination, the noon / one o’clock gun based on Dawes determination probably continued to be used for setting chronometers, even after Karl Runker’s precise determinations of latitude and longitude at Governor Brisbane’s private observatory in Parramatta Park in 1821.

Sir Thomas Mitchell adopted the Parramatta Park determination for his expeditions and for his 1834 “Map of the Nineteen Counties” and would have noticed the position difference with Dawes when drawing his map. He also commented on his disparity with the Admiralty Chart positions when he reached the Bass Strait on his Third Expedition in 1836.

On 16 December 1824, Captain William Hovell of the Hume and Hovell Expedition believed he reached Western Port, as planned. Two years later the government decided to settle at Western Port but, based on Hovell’s longitude, landed at Corio Bay instead, found the land unsuitable and had to abandon the settlement. Hovell’s longitude was 80 km out of position.

An additional observation was made in 1840 (Figure 1):

- 1840 Surveyor Charles Tyers determination of the longitude of Fort Macquarie (4): His determination of longitude was 151º15’14” E differing by 0º02’54” and 4.5 km east of actual (151º12’20”). This resulted in part for the 141º E meridian border between Victoria and South Australia being set and marked about 3 km west of the meridian.

It is unknown when Dawes determination was discontinued and was replaced, and with what. The Dawes Point observatory itself had disappeared by 1795. The time ball was relocated to Fort Phillip / Signal Station (at the present-day Observatory site) in 1825, although Fort Macquarie (at the present day Opera House site) was apparently used from 1817 until when the Sydney Observatory was built in 1858. The one o'clock gun was relocated from the Dawes Point Battery to Fort Denison in 1906.

Additional observations were made from Sydney Observatory in later years (Figure 1):

- 1859 longitude (5) was determined at 151º14'59" E, located in the middle of Sydney Harbour, half way to the Heads – even with good equipment, astronomers were having problems 70 years later.
- 1883 (6), when time signals could first be transmitted to Sydney by cable from Greenwich, the determination of Longitude was 151º12'22" E.
- 1903, using both east and west cables from Greenwich, longitude was determined as 151º12'20" E.
- At Station "E" (7), used for trigonometrical surveys, longitude was determined as 151º12'23.1" E.
- In the 1986 edition 1:25,000 AGD66 Parramatta River topographic map, Observatory Hill, Sydney latitude was scaled at 33º51'39.5" S (200 m south of actual), longitude at 151º12'12.1" E (115 m west of actual) and magnetic declination (1980) shown at 12º12' E.
- In the 2002 edition 1:25,000 GDA94 Parramatta River topographic map (8), Observatory Hill, Sydney latitude was scaled at 33º51'34" S, longitude at 151º12'16" E, and magnetic declination (2002) shown at 12º36' E.

Table 1 summarises these geographical positions and their difference from the actual GDA94 position (also see Figure 1). It should be noted that the Greenwich Time Signal (GTS) pips by radio commenced in 1924. The longitude determinations for places like Bathurst were dependent on the Sydney longitude determination and, as mentioned earlier, these varied considerably over time.

Table 1: Determinations of the geographical position of Sydney.

No.	Determination	Method	Latitude (S)	Diff	Longitude (E)	Diff
1	1788 Dawes	Lunar distance	33º52'39"	+1'04" (1.9 km S)	151º19'30"	+7'14" (11.2 km E)
2	1788 Hunter	---	33º51'30"	-0'05" (150 m N)	151º13'	+0'44" (1.1 km E)
3	1803 Flinders	---	33º51'45"	+17.3" (300 m S)	151º11'49.5"	-0'26.5" (675 m W)
4	1840 Tyers	Chronometer	---	---	151º15'14"	+2'58" (4.6 km E)
5	1859	Astronomy	---	---	151º14'58"	+2'42" (4.18 km E)
6	1883	Time signal (cable)	---	---	151º12'22"	+0'06" (155 m E)
7	Station "E"	---	33º51'41.1"	+6.1" (190 m S)	151º12'12.1"	+0'03.9" (115 m E)
8	2002	---	33º51'35"	---	151º12'16"	---

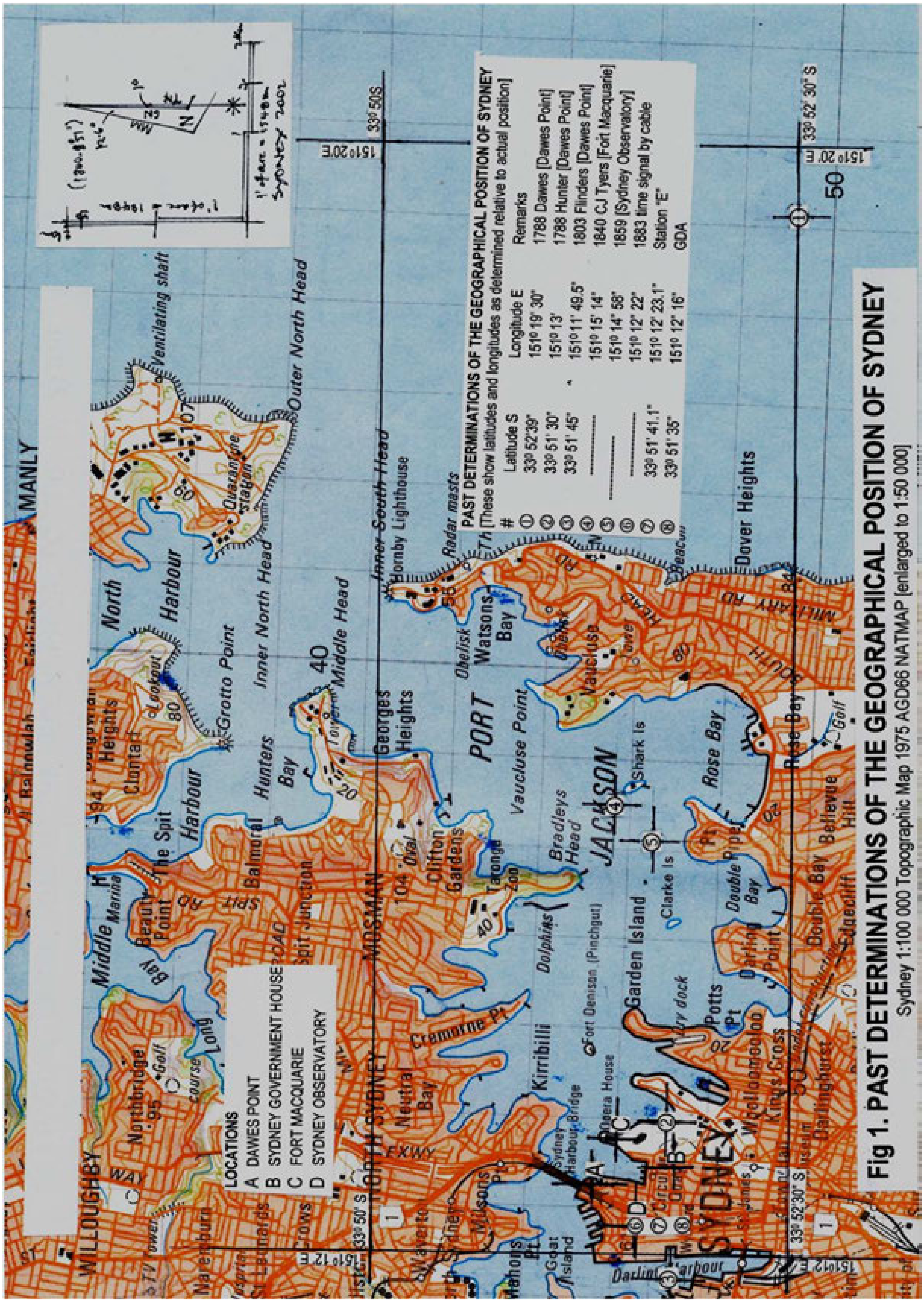

Figure 1: Past determinations of the geographical position of Sydney.

5 THE LATITUDE AND LONGITUDE OF THE BATHURST FLAG STAFF

5.1 The Present Position of the Bathurst Flag Staff

The position of the Bathurst Flag Staff was re-determined, not from astronomical observations, but by the scaling of its plotted position from early surveys and then its relocation from survey evidence relative to the present layout of the City of Bathurst. The present 2014 re-determined position of the Bathurst Flag Staff was determined by Surveyor Michael Spiteri using Global Positioning System (GPS) technology to be at latitude 33º24'47.176" S and longitude 149º35'08.6526" E (GDA94). This is referred to in this paper as the *actual* geographical coordinate position for the comparison of historic determinations. The magnetic declination was 11º54' E (2009). The actual position of Sydney Government House (Bridge Street) is latitude 33º51'49" S and longitude 151º12'40" E (GDA94), and the bearing and distance between it and Bathurst is 288º17'36" and 158.796 km for actual polar coordinate comparisons.

5.2 The 1815 Official Position of the Bathurst Flag Staff

The 1815 official position is contained in a dispatch by J.T. Campbell, Governor Macquarie's Secretary (see section 2). Because the stated geographical coordinates and the stated distances vary significantly in themselves, it is the author's contention that the coordinates may have been viewed "by observation taken at a Flag Staff" by Oxley and that the stated dimensions were calculated from Evans' traverse. Otherwise, why were the differing distances stated? From the precision of the latitude and relative imprecision of longitude, it is assumed that the stated coordinates were based on lunar distance observations. Because the public notice of the Flag Staff event also refers to the road distance, it tends to support the contention that the stated distances were determined from Evans's traverse. Figure 2 illustrates the past determinations of the geographical position of Bathurst, which are discussed in the following sections.

5.3 Dead Reckoning Coordinate Dimensions: (1) in Figure 2

The stated Northing (latitude) is 27½ miles being 44.247 km, and the stated Easting (departure) is 94½ west being 152.05 km. The stated origin is Government House, Sydney, being determined at 33º51'49" latitude and 151º12'40" longitude (GDA94). Using Evans' coordinates, the position at Bathurst is latitude 33º27'51" and longitude 149º34'19", being about 5.6 km south of actual and 1.28 km west of actual. The longitude is surprisingly close considering the method used. Bearing and distance is 286º13'31" and 158.357 km, being 2º04'02" and (only) 440 m less than actual (see further comment on difference in magnetic variation below).

George William Evans traversed between Emu Ford and Bathurst from 13 November 1813 to 8 January 1814. Evans was a land surveyor and used a circumferentor and chain. His equipment list did not include a chronometer or a sextant. Evans had (and used) a circumferentor (essentially a compass with sighting vanes to read to the nearest degree) and two Gunther's chains to measure the traverse. All land surveys were on Magnetic North.

In calculating his coordinates, Evans had to adjust his traverse to True North. It is not known what magnetic variation was adopted by Evans, but it probably varied along his traverse at that time (1815) at about, say, 9º47' E at Sydney to 9º05' E at Bathurst. However, compared

to actual, Evans' magnetic variation is a surprising 2º04' less than it should have been; was Dawes' 1788 magnetic variation of 7º54' E adopted?

The connection between Sydney Government House and Emu Ford that is used in the coordinate calculation is also not known. However, the Surveyor General's Office maintained a 'charting map' on which additional survey information, as it became available, was plotted relative to the meridians and True North. Maybe it was scaled from this map.

5.4 Stated (Polar) Bearing and Distance: (2) in Figure 2

This was stated as "bearing west 20º30' north 83 geographic miles, or 95½ statute miles", being 290º30' for 154 km (geographic) or 153.66 km (statute miles) providing ordinates of 53.8 km north and 143.9 km west. This position differs significantly with the before mentioned stated coordinate position (i.e. 9.6 km north-south and 8 2 km east-west) and is 4 km north and 6.9 km east of actual. It is 2º12' more and 5 km less than actual bearing and distance. Its basis, and why it was stated, is not known, and it is not very helpful.

5.5 Stated 1815 Latitude and Longitude: (3) in Figure 2

The stated latitude (33º24'30") is very close to actual (33º24'47.18"), being about 590 m further north. This is indicative of the greater precision in sextant observations for latitude. The stated longitude of 149º37'45" is 4 km east or short of actual (149º35'09"), which, plotted today, is located east of Kelso and towards the Bathurst Aerodrome. Comparatively, Evans Easting is closer (being 1.28 km west), indicating greater precision by dead reckoning. Bearing and distance is 288º57'55" and 155.163 km, being 0º40'17" more and (significantly) 3.633 km shorter than actual.

Unlike Evans, Surveyor General John Oxley was a navy man and had a sextant and an artificial horizon. He requested a "timepiece" or chronometer in March 1815 and may not have received it by 5 May, when the Flag Staff was erected. Irrespective of this, he probably used lunar distances for his longitude determination (hence the imprecision).

Considering the methods used 200 years ago, and using a sextant for latitude determination and dead reckoning for longitude, the results are very close to actual. A major unknown is the difference with his longitude determinations then, and those two years later. Table 2 summarises the polar coordinate comparisons of the stated positon of Bathurst.

Table 2: Polar coordinate comparisons of the stated position of Bathurst from Sydney.

No.	Determination	Bearing	Diff	Distance	Diff
1	Dead reckoning	286º13'30"	-2º04'	158.357 km	-440 m
2	Bearing & distance	290º30'	+2º12'	154 km	-5 km
3	Latitude & longitude	288º57'55"	+0º40'20"	155.163 km	-3.633 km
8	Actual	288º17'36"	---	158.796 km	---

5.6 The 1817 Determination by John Oxley: (4) in Figure 2

In 1817, Surveyor General John Oxley located the Flag Staff at latitude 33º24'30" S and longitude 149º30' E. The latitude is the same as the 1815 determination. However, the longitude is significantly west, near Mount Stewart, at a distance of 7.9 km west, and not east, of the actual/present location. This means the distance between the 1815 and 1817 determinations of longitude is over 12 km, a more significant difference. Was this because he

used a chronometer in 1817? Was it set to the noon gun/time ball at Dawes Point and was this set to Dawes' longitude of 151º19'30"?

An interesting theory – see (5) in Figure 2 – is that Oxley's difference between Sydney and Bathurst is 1º41'30", i.e. 151º11'30" – 149º30' = 1º41'30" according to Whitehead (2004, p. 21), which, when deducted from Dawes' Sydney determination of 151º19'30" gives a revised Bathurst longitude of 149º38'. This is almost the same as the 1815 determination of 149º37'30". Taking this further, 149º30' (see Oxley's 1822 chart) is 0º05'05" west of actual which coincides generally with the average difference of his longitudes (i.e. 0º08'10" west) with actual longitudes on his 1818 journey. Interestingly, Whitehead (2004, p. 58) suggests that he may have changed his Port Macquarie longitude to agree with a later (second) observation used when compiling his 1822 chart. Did he find Dawes' determination was 0º8' out and change Sydney as well?

5.7 The 1829 Determination by Mitchell: (6) in Figure 2

Major (later Sir) Thomas Mitchell, Surveyor General, plotted the position of Bathurst (and, by location, the Flag Staff) on his 'Nineteen Counties Map'. Mitchell used his rapid mapping trigonometrical survey method – he wrote an army manual on the subject (Mitchell, 1827) and used it in the mapping of the 'Nineteen Counties'. He used measured baselines, theodolite triangulation between mountain tops and then, circumferenter and chain infill of detail. His origin of latitude and longitude was latitude 33º48'50.68" S and longitude 151º01'48" E being the centre of the transit at Governor Brisbane's Parramatta Observatory. He did not use Dawes Point or Dawes' values. His baselines were at Botany Bay and Lake George, with lesser bases at Mittagong and elsewhere for local purposes and as checks on adopted distances. His latitudes were based on sextant observations "on Peel River, at Newcastle, Warawolong, Lake George, Jumley (Lumley), and Kurradacbidgee."

Mitchell's Nineteen Counties Map has a constant scale between latitudes and differing scales between lines of meridians to allow for their convergence, thereby maintaining right angles between his parallels of latitudes and his meridians of longitude. From the plotted infill in the map by Surveyor James Richards in 1828-29, the Bathurst Flag Staff position scales at latitude 33º25'40" S and longitude 149º37'30" E. This is 1.6 km south and 3.67 km east of the 2014 and actual location of the Flag Staff.

It should be noted that the 1815 longitude (149º37'30" E), Oxley's 1817 determination, as revised (149º38') and Mitchell's 1829 determination (149º37'30"), using three different methods (lunar distance, chronometer and trigonometrical survey), are virtually the same. Yet they are all about 0º02'40" or 4 km east of actual… Why?

5.8 1985 Topographic Map Determination: (7) in Figure 2

Using the 1:25,000 Bathurst topographic map (AGD66), the Flag Staff position was scaled at 33º24'52.7" S and 149º35'05" E, being 170 m south and 95 m west of actual.

5.9 2014 GPS and GDA94 determination: (8) in Figure 2

Based on position plotted on early survey plans, the Flag Staff, as re-determined, was surveyed by GPS at (actual) latitude 33º24'47.176" S and longitude 149º35'08.6526" E (see section 5.1).

In conjunction with Figure 2, Table 3 summarises the determinations of the geographical position of the Bathurst Flag Staff.

Table 3: Determinations of the geographical position of the Bathurst Flag Staff.

No.	Determination	Method	Latitude (S)	Diff	Longitude (E)	Diff
1	1815 Evans	Dead reckoning	33º27’51”	+3’03” (5.6 km S)	149º34’19”	-0’50” (1.28 km W)
2	1815 -------	Bearing & distance	33º22’40”	-2’06” (3.7 km N)	149º39’39”	+4’31” (6.9 km E)
3	1815 Oxley	Lunar distance	33º24’30”	+17.3” (543 m S)	149º37’45”	+2’36.4” (4.03 km E)
4	1817 Oxley	Chronometer	33º24’30”	+17.3” (543 m S)	149º30’	-5’08.6” (8 km W)
5		(revised)	33º24’30”	+17.3” (543 m S)	149º38’	+2’51.3” (4.43 km E)
6	1829 Mitchell	Trig. survey	33º25’40”	-52.8” (1.63 km N)	149º37’30”	+2’21.4” (3.67 km E)
7	1985 AGD66	Map scaled	33º24’52.7”	+5.5” (170 m S)	149º35’05”	-3.65” (95 m W)
8	2014 GDA94	GPS survey	33º24’47.17”	---	149º35’08.65”	---

6 MAP PROJECTIONS, FIGURES OF THE EARTH AND GNSS

Map projections and the figure of the earth have varied with improvements in data, overseas mapping trends and mathematic calculation. Initially, Mitchell adjusted for the convergence of meridians and the maintenance of right angles between meridians and parallels of latitude by varying the scales between each parallel of latitude. In about 1858, the colonies of Australia followed the Ordinance Survey of Great Britain by using Clarke’s geocentric 1858 figure of the earth and used Cassini-Soldner and Transverse Mercator projections. During the 1960s to 1990s, Australia adopted the Australian Geodetic Datum (AGD66) to determine the best fit with the continent. In the 1990s, with the advent of GPS, Australia reverted to a geocentric datum, the Geocentric Datum of Australia 1994 (GDA94), for the best fit for Global Navigation Satellite System (GNSS) observations (ICSM, 2009). The mapping projection for GDA94 is the Map Grid of Australia (MGA94), replacing the Australian Map Grid (AMG) but both are standard 6º Universal Transverse Mercator (UTM) projections used by all states and territories across Australia. The Surveyor General of NSW has also endorsed the use of a GDA94 Lambert Conformal Conic projection for state-wide Geographical Information System (GIS) data in NSW.

It should be noted that the GPS location is about 30º and 195 m from the location of the Flag Staff on the Bathurst 1:25,000 topographic map (1985). This is due to the change from AGD66 to GDA94, the latest determination of the figure of the earth so as to be compatible with GNSS technology. The Australian tectonic plate is moving NNE (about 22º30’) at a rate of about 60 mm each year. It also rotates (J. Haasdyk, pers. comm.): the west coast (Perth, 70 mm/yr) is moving more than the east coast (Sydney, 57 mm/yr). Since the Bathurst Flag Staff was erected 200 years ago, it (and the Australian tectonic plate) is estimated to have moved about 11.7 m in the NNE direction.

While, in reality, the celestial sphere remains constant and the tectonic plate moves, for practical, legal and continuity reasons its position on the ground, on the Australian tectonic plate and GDA94, is fixed. For the same reasons, measurements and dimensions are ground (level) distance where they are measured and used rather than projection distances on MGA94. Similarly, the already established heights relative to the Australian Height Datum (AHD71 on mainland Australia) and AGD66 are used rather than GNSS ellipsoidal heights. For instance, all heights and contours shown on current GDA94/MGA94 topographic maps are referred to AHD71. In this regard, Bathurst has a geoid-ellipsoid separation of about 25.2 m, i.e. the AHD71 zero surface is located above the ellipsoid (J. Haasdyk, pers. comm.).

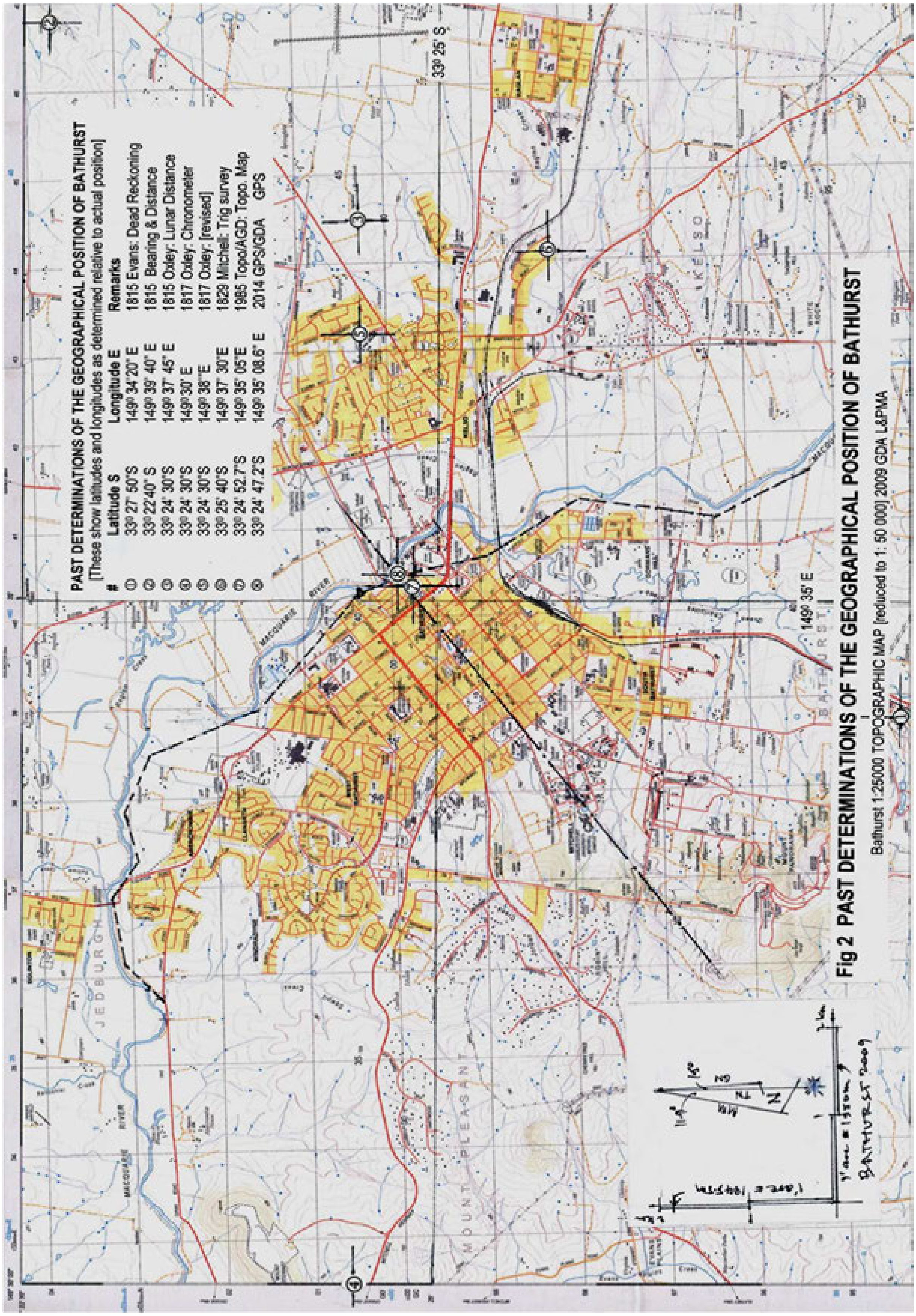

Figure 2: Past determinations of the geographical position of Bathurst.

Global GNSS users utilise a dynamic datum (International Terrestrial Reference Frame, ITRF) with moving tectonic plates, rather than a datum fixed to the tectonic plate such as GDA94. For instance, ITRF positions in Australia have already moved about 1.2 m in relation to GDA94 since 1994. This prompts the possibility of a revised Australian datum in the near future as GNSS precision becomes better and better and more precise data becomes available.

7 DETERMINATION OF THE LOCATION OF THE FLAG STAFF AT BATHURST

There is obviously a disparity in the various Bathurst Flag Staff position determinations, which were obtained in many ways and affected by the evolution or refinement of surveying and geographical coordinate determination generally and in Australia. It is fascinating that, although its apparent purpose was for the determination of absolute geographical position, it is clearly evident that because of past imprecision and uncertainty, these determinations are practically useless in relocating the present-day position of the Bathurst Flag Staff. So, to re-determine its location, we must turn to surveyors and, especially, the forensic aspects of cadastral surveying.

7.1 Background

Cadastral (registered) surveyors look back at past survey documentation, plans and marking in order to determine the present-day boundary position on the ground. In this instance, it certainly helped that the Flag Staff was the origin or reference point for the setting out of the Town of Bathurst. However, in the absence of survey field notes and survey dimensions, the method used to relocate the Bathurst Flag Staff was the correlation, by scale, of its position plotted on copies of the original or early survey plans. Four survey plans with the Flag Staff plotted on them from survey were used (Figure 3):

- SR1293 Evans (possibly 1815).
- Evans (possibly 1818).
- SR1294 Meehan (probably 1821).
- Mitchell (Town Plan 1833).

In the context of Figure 3, the following should be noted:

- Plans (a) and (b) show Governor Macquarie's town layout. Plan (d) shows Governor Darling's (and current) town layout.
- The scales of the plans were determined by the ratio of the town block dimensions, Plan (c) graphical scale, being divided by the actual scaled distance.
- The dimensions to the Flag Staff were determined by multiplying the actual scaled distances by the scale ratio.
- The Flag Staff dimensions were related crosswise to the centreline of the present-day town blocks between George and William Streets. Longitudinally, the distance to the SW side of Stanley Street was determined by deducting 362.1 m (18 chains) from the scaled dimensions to the SW side of Durham Street.

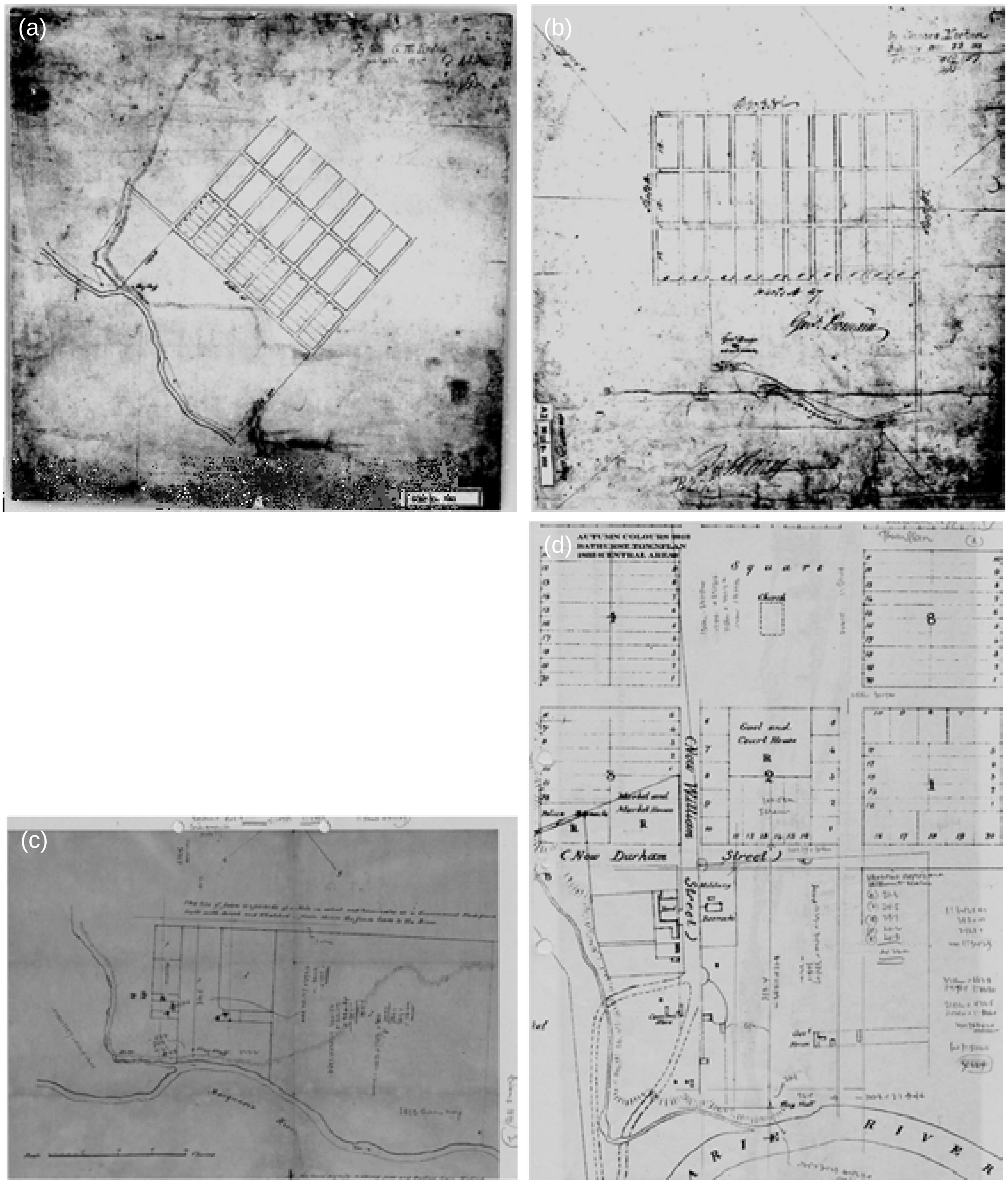

Figure 3: The four survey plans used in determining the original location of the 1815 Bathurst Flag Staff: (a) SR1293 Evans 1815, survey of Government Domain, (b) SR1294 Meehan (possibly 1821), survey of Government Domain, (c) Evans (possibly 1818), and (d) Mitchell 1833.

On 9 December 1813, Surveyor George Evans placed a survey traverse mark remarkably close to, if not on, the site on which the Bathurst Flag Staff was erected 1½ years later, on 5 May 1815. The Flag Staff is clearly shown on the traverse mark in the surveys by Evans and Meehan. This is connected by plot to the ‘Macquarie’ planned layout of Bathurst.

Initially, the Bathurst town layout had half-acre lots with one-chain frontages, 2½ chain depths, and roads one chain wide. This town layout was used by surveyors Meehan and Evans and is shown on their plans. The town blocks were 12 chains by 5 chains with 1 chain wide roads, i.e. 12 by 1 chain wide lots between side streets and 2 x 2½ chain depths, being five chains, between the longitudinal streets. The Surveyor General’s Office and Governor Macquarie had probably approved this town layout.

In Colonial NSW, a 3-step process was usually applied in the designing and surveying of early towns. The practice was:
1. The surveyor to undertake a feature survey of the town site and submit it to the Surveyor General's Office.
2. At the Surveyor General's Office, a town design was prepared.
3. This was sent back to the (or a) surveyor to set out the town.

It seems that this was done initially, as Evans and Meehan both plot the same Bathurst town layout. Subsequently, it is known that Governor Darling regulated a change in town layouts, and this is probably why Mitchell's 10 chain by 10 chain town blocks with 1½ chain wide roads replaced the previous layout. This is the 'Darling' and present Bathurst town layout. In order to compare the four surveys, it was necessary to identify anything common between the two different Bathurst town layouts. In the absence of any other features, the one common feature was the Flag Staff.

The Flag Staff was erected on Friday, 5 May 1815 and at the flag raising on Sunday, 7 May, Governor Lachlan Macquarie proclaimed "the erection of a town at some future period" to be named "Bathurst". However, while the Flag Staff was central to the later Mitchell town layout, the questions are:
- Was it also central (or not) to the initial Evans and Meehan Town Blocks? If so, which ones?
- Was there a common crossroad adopted in both town layouts? If so, which road? Which side? Was it 1 chain or 1½ chain wide from Evans' fence, or is it the centre?

7.2 Presumptions

In order to compare the surveys, it is necessary to correlate the initial Evans/Meehan/'Macquarie' town layout with the Mitchell/'Darling'/present town layout. Prompted by field discussions on 09/12/2013, and by the receipt of new copies of the survey plans, the layouts were compared using the common location of the Flag Staff and the centre of the town blocks generally. From this, the following three presumptions were then made.

Presumption 1: *The centreline of the 5 chain wide (initial) and 10 chain wide (present) town blocks between William and George Streets were projected from the Flag Staff*. The presumption is that the town of Bathurst was set out by Mitchell from this centreline. The centreline between Mitchell's 10 by 10 chain town blocks was correlated with the 5 chain wide town block layouts on the Meehan and Evans surveys. It was found that the centreline coincides precisely with the centreline of the third town block on Meehan's plan and the first town block on Evans's plans. Their (Evans, Meehan and Mitchell) location of the Flag Staff is on the extension of the centreline of these town blocks. This is indisputable. In a survey sense, this centreline could be termed an 'azimuth' used in subsequent surveys. Surveyor Michael Spiteri has recently re-determined the centreline of the William and George Street town blocks at the centre of Stanley Street and bearing 51º31' TN from the town.

Presumption 2: *The SW side of Durham Street is common to both town block layouts*. The fence shown on Evans' 1818 survey seems to form the NE boundary of Durham Street. However, this would then put the Flag Staff 1.4 chain (28 m) from the SW boundary of Stanley Street, contrary to all other evidence. Instead, it appears that it conformed with the initial (Macquarie) 1 chain wide road layout that was subsequently widened to the (Darling) 1½ chain wide road layout of Mitchell's plan and the current own layout. Therefore, it is presumed that the SW side of Durham Street must be common to both town layouts.

Presumption 3: *The SW side of Stanley Street was determined at 18 chains (362.1 m) from the SW side of Durham Street*. This was determined by original dimension, being the 16.5 chain town block depth plus the 1½ chain width of Durham Street. This was checked generally with Google Maps, which scaled at 364.5 m at George Street and 358.5 m at William Street, the average being 361.5 m. This confirms generally with the 362.1 m or 18 chains dimension adopted. The SW side of Stanley Street, bearing 141º52'30" TN, as redefined by Surveyor Michael Spiteri, becomes the legal abuttal for the redefined, relocated original site of the Flag Staff.

7.3 Scaling of Plotted Positions

The lack of stated survey dimensions from the SW side of Durham Street to the Flag Staff means that precision scaling and mathematical proportion must be applied to copies of the four survey plans by surveyors Evans, Meehan and Mitchell. The method applied is that used to determine and check scale ratios, dimensions and measurements on copies of plans and maps enlarged or reduced by computer and/or photocopying processes.

The probability of determining the most likely position of the Flag Staff is improved or enhanced by:

- Use of the oldest surveys. It should be noted that photocopies were used; scaling from the original survey plans would have given more certainty.
- Averaging the position from four plans of survey and their mutual validation.
- Averaging the position from surveys by the three surveyors.

However, this methodology has the following drawbacks:

- The comparatively small size and scale of the plans used and that one millimetre at 1:5,000 scale ratio (average scale of the plans) represents 5 m on the ground.
- The precision of the initial plot of the surveys.
- The use of copies of the originals and the variable paper and copier shrinkage/enlargement (e.g. the scale ratios of Mitchell's plan are 1:4,742 across and 1:4,785 lengthwise).
- The precision of the scaling in the determination of the plan scale ratios and the scaled distance (a number of scale readings were taken for averaging and as a check).

The method applied involves four steps (Figure 4):

1. Determining the scale ratio of the plan. This was calculated by dividing the stated or known dimensions of the town blocks, or the graphical scale on the plans, by their scaled distance. A 1:100 scale and a magnifying glass were used to read fractions of a mm, e.g. dimension 603.5 m (30 chains) divided by scaled measurement of 0.1433 m (143.3 mm) = 4,211, resulting in 1:4,211 scale ratio. This was repeated lengthwise and crosswise, weighted and averaged to, say, 1:4,220 in this example (SR1293).
2. Next, determining the distances/dimensions to the Flag Staff. The distances scaled from the plans were multiplied by the scale ratio of the plan, e.g. (a) crosswise, scaled distance, 0.0113 m (11.3 mm) multiplied by the scale ratio 4,220 = 47.7 m to Flag Staff, and (b) longitudinally, 0.0903 m (90.3 mm) x 4,220 = 381 m from the SW side of Durham Street to the Flag Staff.
3. Determining distance comparisons and differences. The scaled distances were compared with (crosswise) the dimension to the centreline, and (longitudinally) the dimension between the SW sides of Durham and Stanley Streets, e.g. (a) crosswise, 47.7 m – 50.3 m (2.5 chains) = +2.6 m being right of the centreline, and (b) lengthwise, 381 m – 362.1 m (18 chains) = 18.9 m from the SW side of Stanley Street.

4. Determining average or mean position. The distances from the Flag Staff to the centreline and to the SW side of Stanley Street were averaged to determine the best fit as to where the Flag Staff was originally located.

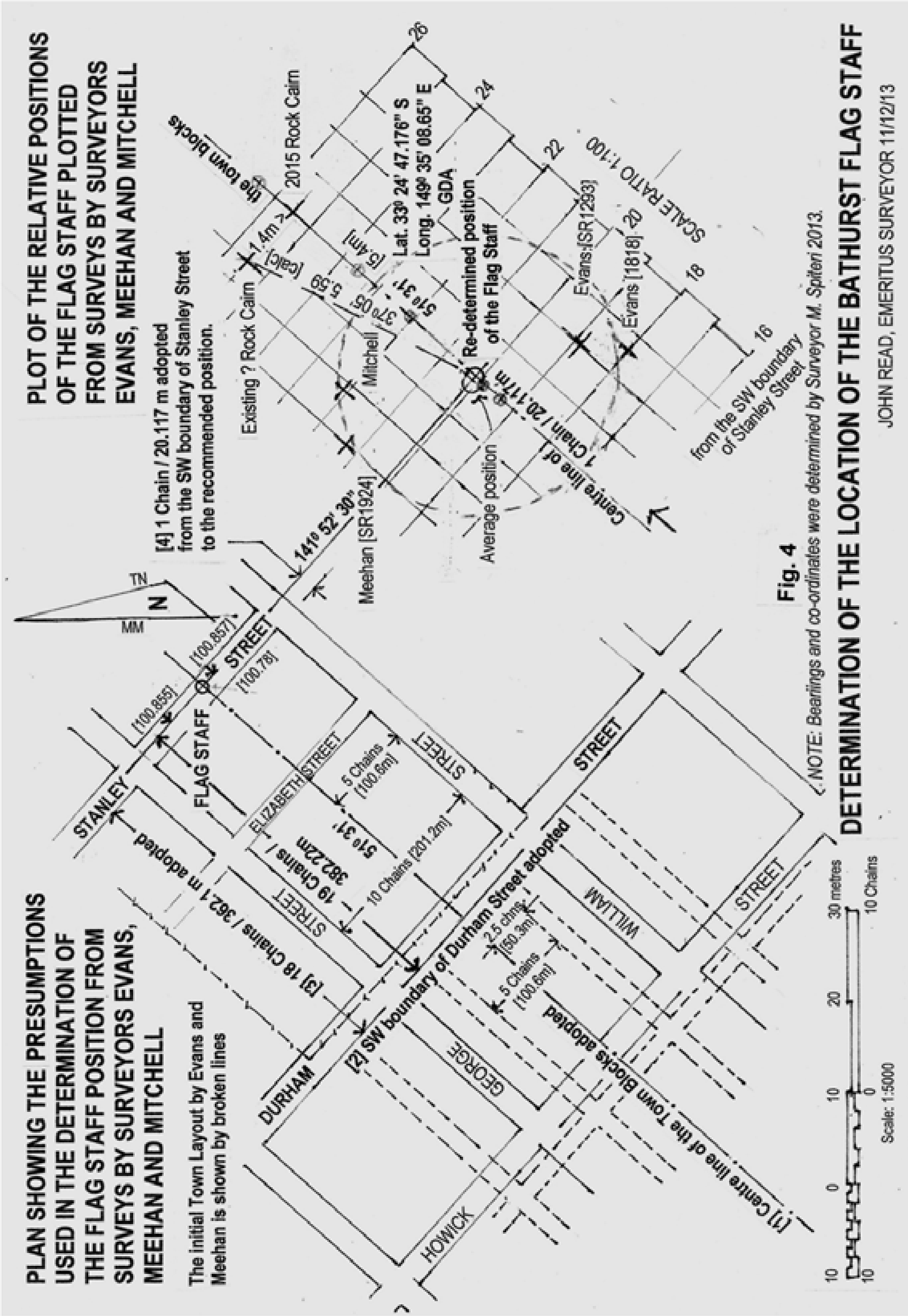

Figure 4: Determination of the location of the Bathurst Fag Staff.

The average position determined from the four survey plans is shown in Table 4. In order to give equal weight to the determination by the three surveyors, the positions on the two Evans plans (i.e. SR1293 and the 1818 Evans plan) were averaged (Table 5).

Table 4: Average position determined from the four survey plans.

No.	Survey Plan	From Centreline	From SW side of Stanley St
1	SR1293 (Evans 1815)	+2.6 m	18.9 m
2	Evans 1818	+2.1 m	18.0 m
3	SR1294 (Meehan 1821)	-3.0 m	21.2 m
4	Mitchell 1833	-2.1 m	22.1 m
	Average Position (1)	-0.1 m	20.1 m

Table 5: Average position as determined independently by surveyors Evans, Meehan and Mitchell.

No.	Surveyor	From Centreline	From SW side of Stanley St
1&2	Evans 1815 & 1818	+2.35 m	18.45 m
3	Meehan 1821	-3.0 m	21.2 m
4	Mitchell 1833	-2.1 m	22.1 m
	Average Position (2)	-0.9 m	20.6 m

7.4 Discussion of Results from the Scaling of Plotted Positions

Considering the presumptions, drawbacks and precision of scaling, the results are surprisingly close. The average and re-determined positions are within 3 m and about 0.5 mm by scale of all four plotted positions. All lengthwise determinations come within 4 m on the ground (18.0 m to 22.1 m) and within 1 mm at the 'average' plan scale used of 1:5,000, and crosswise within 5.6 m (-3.0 m to +2.6 m of the centreline). This is well within the order of accuracy and precision expected in this method of redetermination.

The average positions crosswise (1) -0.1 m and (2) -0.9 m confirm the assumption of the town block centreline. Average position longitudinally of (1) 20.1 m and (2) 20.6 m across Stanley Street are 'fortuitously' and coincidentally close to 20 m and 20.117 m or one chain, and certainly nowhere near the 25.5 m to the existing undated rock cairn (see section 7.4.2).

Back in the 1800s, chains and links (rather than metres) were the official land measurement unit, so 'rounding off' to one chain, rather than to 20 m, makes more sense. The average position (1) of the four plans is practically 'spot on' both ways. Thinking about this logically, and that the determination of position is already based on three presumptions, a fourth presumption is suggested:

Presumption 4: *The SW boundary of Stanley Street was originally located one chain (20.117 m) from the Flag Staff, and conversely, today, the Flag Staff should be relocated on the centreline at a distance of one chain (20.117 m) NE from the SW side of Stanley Street.*

7.4.1 Town Maps

Using the same scaling method, the Flag Staff from the SW side of Stanley Street scales variously on the four town maps as 19.3 m (undated), 19.9 m (undated), 26.4 m (1860) and 23.6 m (1897), i.e. on average 22.3 m. Although 2.2 m longer, it generally confirms the scaled determination of position from the early survey plans. In any dispute between them (i.e. the old surveys and the later town maps), greater regard must be given to the scaling of plots on the oldest surveys rather than those on the later town plans probably derived from them.

7.4.2 Existing Rock Cairn Monuments

Bathurst has two rock cairn monuments:

- The 1930 Proclamation Cairn on the corner of William and Stanley Streets states that "on this spot Governor Lachlan Macquarie proclaimed the Town of Bathurst", which implies that it was the site of the 1815 Flag Staff and the Proclamation of the Town of Bathurst. This was subsequently proven to not to be the case.
- The undated, unacknowledged and unsubstantiated rock cairn with a bronze plaque on top states that "this plaque marks the exact location of the Flag Staff". However, again, this has been proven not to be the case. Its position from a survey by Surveyor Michael Spiteri is (TN) 37º05' and 5.59 m (calc.) from the re-determined Flag Staff site (and 25.5 m from the SW side of Stanley Street).

The origin of the undated rock cairn is obscure and the survey basis for its location is unknown. Hence (especially now that it has been conclusively proven otherwise) any claim that it is the "exact location" of the 1815 Flag Staff lacks any and all credibility. Apparently aware of this, this rock cairn is now to be moved 1.4 m sideways (now being referred to as the 2015 Rock Cairn by this author) to, at least, line up with the centreline of the town block. However, it will *not* be moved 5.4 m towards town to the re-determined site.

On the site (and at great cost) Bathurst Regional Council is constructing its 'Bicentennial Project and Re-instalment of Flag Staff' proposal consisting of a 500 m^2 suspended concrete slab centred on the relocated 2015 Rock Cairn. This will be viewed through a glass pyramid with the replica of the 1815 Flag Staff erected and floodlit overhead. These will be the focal point of the Bathurst bicentenary celebrations in 2015. Unless corrected, the plaque on the 2015 Rock Cairn will still claim to be the "exact location" of the Flag Staff when clearly it is not, and when the re-determined location is actually 5.4 m closer to town.

On the site (and at great cost) Bathurst Regional Council is constructing its 'Bicentennial Project and Re-instalment of Flag Staff' proposal consisting of a 500 m^2 suspended concrete slab over an amenities block and centred on the relocated 2015 Rock Cairn. The cairn will be viewed through a glass pyramid with the replica of the 1815 Flag Staff erected and floodlit overhead. These will be the focal point of the Bathurst bicentenary celebrations in 2015. Unless corrected, the plaque on the 2015 Rock Cairn will still claim to be the "exact location" of the Flag Staff when clearly it is not.

8 CONCLUDING REMARKS

The re-determined position of the 1815 Flag Staff is located on the extension of the centreline of the town blocks between William and George Streets and at a distance of one chain (20.117 m) NE from the SW side of Stanley Street. This was rationalised from presumptions made regarding its position relative to the town block layouts, and by the scaling of the plotted Flag Staff position on early plans of survey by surveyors Meehan, Evans and Mitchell. Its position has been determined from the survey by Surveyor Michael Spiteri at latitude 33º24'47.176" S and longitude 149º35'08.65" E in GDA94 and it is located (TN) 231º31' and 5.415 m towards Bathurst from the proposed 2015 Rock Cairn and replica Flag Staff.

The 1815 Bathurst Flag Staff was placed as a reference point for the survey of the Town of Bathurst and for the exploration and survey of the Central and Western Districts of NSW.

Today, the highlighted 2015 Rock Cairn and the replica Flag Staff are a unique reminder of this function and the importance of surveying monuments to the community generally and to the surveying profession in particular. However, it is a pity that the Rock Cairn, claiming to be at the "exact location" of the Flag Staff, is being relocated to the wrong spot.

ACKNOWLEDGEMENTS

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Survey Infrastructure Preservation and Upgrade: Trigonometrical Stations in NSW

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ABSTRACT

Land and Property Information (LPI) is the custodian of the geodetic infrastructure in New South Wales, which incorporates approximately 6,000 traditional trigonometric stations that formed the backbone of the survey control network before the introduction of more than 160 CORSnet-NSW stations. Keeping the ageing passive geodetic infrastructure current and ready for utilisation requires regular maintenance such as fit-for-purpose assessments, verification of owner records, contact details, access details, and (most importantly) recent, time-stamped Global Navigation Satellite System (GNSS) observations. As this level of maintenance is neither viable nor justifiable for the entire network of trig stations in the modern era, LPI has selected a subset of 'high-scoring' trig stations based upon a number of factors including a 'TrigStar' rating, monument quality and significance within the contexts of survey network, local community and heritage. After the successful completion of a campaign-style pilot project in the Central West, and subsequent campaigns on the South Coast, in Northern NSW and in Southern NSW, LPI continues to carry out this trig rationalisation and targeted upgrade across the State. LPI plans to complete its first pass of trig maintenance and upgrade before the realisation of the nation's next-generation datum in 2015. This paper outlines how this method provides an effective strategy to maintain geodetic infrastructure across NSW, connect traditional survey infrastructure with modern-day satellite-based positioning and contribute new GNSS observations towards the next-generation Australian datum.

KEYWORDS: *Survey infrastructure, trig stations, preservation, maintenance, AUSPOS, datum improvement.*

1 INTRODUCTION

Land and Property Information (LPI) is the custodian of the survey control network in New South Wales (NSW). Currently, some 250,000 marks are managed within the Survey Control Information Management System (SCIMS), the State's survey control database (Kinlyside, 2013). This includes approximately 6,000 traditional 'passive' trigonometric (trig) stations and more than 160 'active' trig stations referred to as Global Navigation Satellite System

(GNSS) Continuously Operating Reference Stations (CORS) as part of CORSnet-NSW (e.g. Janssen et al., 2013; Janssen, 2014).

In this context, it is useful to briefly summarise the history of the geodetic network in NSW (Rassaby, 1980). The Trigonometrical Survey of New South Wales, as it was then known, commenced in 1867 with the selection of the first baseline at Lake George and continued with little interruption for almost 50 years until it was suspended for reasons of economy and war in 1916. By then, about one third of the State (mainly in the south-east) had been covered by a series of well-conditioned triangles of first and lower orders. The survey was resumed intermittently between the two World Wars with much of its progress attributable to the Royal Australian Survey Corps, particularly the connections to the Victorian and Queensland networks, and along the North Coast.

The Division of National Mapping extended the first-order networks eastwards through Broken Hill from South Australia to Cobar (1955-56) and north of Broken Hill into Queensland (1961). Other networks were established by the NSW Department of Lands between Tamworth and Condobolin (1956-57), Cobar and Ivanhoe (1964), and around Narrabri (1957). Together with the first-order traverses performed by the Royal Australian Survey Corps, prior to the national adjustment of 1966, the geodetic network had extended to approximately half of the State – this had taken 100 years.

A turning point in the geodetic survey network was reached in 1973 with the formulation of a plan to update, revise and complete the network to acceptable standards. This systematic rationalisation commenced in the Sydney-Newcastle-Wollongong region and continued through the coastal belt and then westwards. It was found that many of the stations listed in the old County Registers had disappeared as a result of the type of marking used and the difficulty of protection. Wherever possible, the old-style cairn and pole stations were replaced by a concrete pillar with demountable mast and vane, allowing constrained centring of theodolite and distance measuring equipment.

Looking after the State's survey control network is in everyone's interest because survey marks support billions of dollars of investment, property rights and infrastructure. Loss of marks can significantly degrade the integrity of legal property boundaries and spatial infrastructure. LPI champions the preservation of survey marks through its "Survey Marks: All About Protecting Them" campaign (LPI, 2012) and Surveyor Generals Direction No. 11: "Preservation of Survey Infrastructure" (LPI, 2004). This effort is everybody's responsibility.

However, keeping the substantial geodetic component of the survey control network current and ready for utilisation requires a different approach. Regular maintenance such as fit-for-purpose assessments, verification of owner records, contact details, access details, and recent, time-stamped GNSS observations are primarily an LPI responsibility. This paper outlines the campaign-style trig station upgrade methods employed by LPI, providing an effective strategy to preserve the most fundamental component of the survey control network across NSW, connect traditional survey infrastructure with modern-day satellite-based positioning and contribute new GNSS observations towards the next-generation Australian datum.

1.1 What Makes a Trig Station?

Trig stations are the traditional backbone of a classical survey control network and form the primary or highest-order network, from which all other surveys are controlled. Trig stations

come in a variety of forms and structures and usually consist of a primary monument or standpoint surrounded by witness or eccentric marks. The primary monument can take on a multitude of forms, ranging from survey pillars (concrete or steel) to plugs in stone underneath rock cairns or in rare cases galvanised iron pipes or stainless steel rods in soil. It should be noted that all CORSnet-NSW stations are also trig stations – these are known as 'active' trigs. Figure 1 illustrates the large variety of traditional trig station monuments in the NSW survey control network. Sometimes these marks have very high historical significance and are usually located in very prominent locations. Lighthouses, church spires, radio masts and tall towers are other forms of trig stations that are sometimes encountered.

Figure 1: Trig station monuments in NSW. Clockwise from top left: Steel pillar, cairn and astro pillar, concrete pillar, obelisk, concrete pillar on grand cairn, and stonework veneer on concrete pillar.

When the first major GNSS campaigns were initiated in the western part of NSW, the Geodetic Control Register (GCR) and SCIMS were maintained as separate databases. This necessitated the inclusion of occupied Permanent Marks (PMs) and State Survey Marks (SSMs) into the GCR such that those locations could be elevated to the status of trig station – commensurate with the importance of those sites and the technical functionality of the GCR. This creation of new trig station sites also included some original State border survey marks being the remains of wooden posts (e.g. TS7344 Mile Post 181 East).

However, a trig station is often more than just the physical infrastructure. It often includes the entire site or surrounding area, which may cover a significant portion of ground (up to a few hectares) and include remnants of cleared lanes to other distant trigs or marks, and permanent tenure and restrictions over the site to protect it. Trig stations usually occupy the highest and most prominent point in the local area, e.g. a hilltop, tall building or silo. While LPI (or its

predecessors) may have been the first occupant of these areas since white settlement, they are now of prime interest and value to other parties, primarily landowners, developers and telecommunication operators.

Table 1 emphasises the vast variety of trig stations across the State. Almost two thirds of all trig stations consist of a pillar or ground mark located on *private land*. Only one third of trig stations are located on government land, illustrating the importance of having a good relationship with landowners. In order to ensure that this key survey infrastructure is valued and protected by landowners, careful relationship management, face-to-face interaction and active trig maintenance are required by LPI. For one in ten trig stations, the mark type is currently unknown or uncertain, which could easily be rectified through a site visit if it is decided to maintain the trig in question.

Table 1: Selected NSW trig station metadata (categories are non-exclusive and percentages do not sum to 100).

Trig Monument Type	**% of Total**
Pillars	36.1
Pins/plugs/pipes	31.5
Unknown	9.8
Reservoirs	3.9
CORS	2.5
Other SCIMS marks	1.0
Towers	0.9
Lighthouses	0.4
Obelisks	0.2
Silos	0.2
Trig Tenure Type	**% of Total**
Private land	61.9
National parks	14.7
Crown reserves	14.6
Not in NSW	2.7
State forest	2.5
Misc. reserves	0.1
DP Connections	**% of Total**
1	9.0
2	4.6
>2	11.9

The witness or eccentric marks are connected to the primary station through a supplementary 3-dimensional terrestrial survey (Figure 2) and are occupied if it is either not possible or inconvenient to occupy the primary standpoint. For example, unpiling any rock cairn generally introduces lengthy time delays and Work Health and Safety (WHS) issues. Eccentric marks can also be used to monitor any potential movement of the main mark, although this is rarely, if ever done. Often, these marks physically consist of different objects, e.g. expended rifle cartridge in concrete, galvanised iron pipe, rod or numbered/unnumbered SSMs. On a side note, the issue of numbered SSMs that are eccentric to trig stations having two sets of coordinates and metadata in SCIMS (i.e. one as an eccentric mark and another as a unique SSM) is known to LPI and is being corrected.

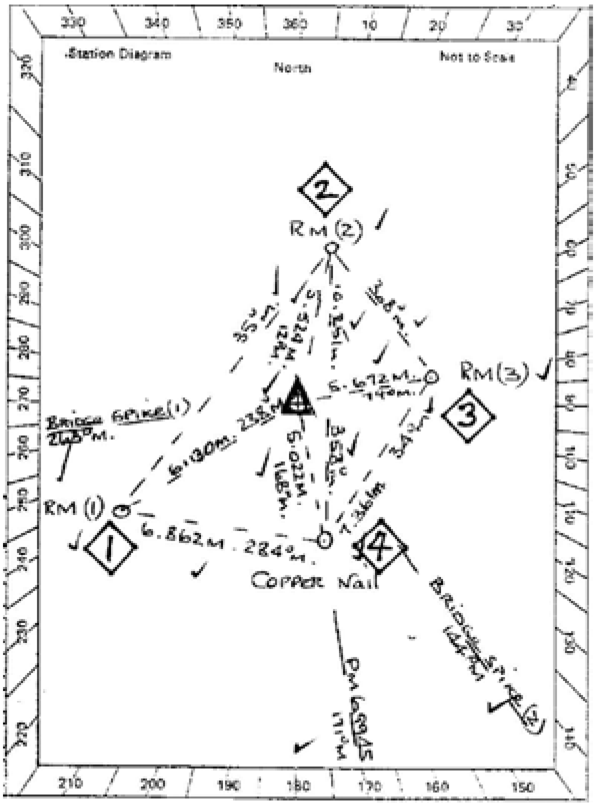

Figure 2: Trig station and witness mark sketch for TS4350 Thoolabool.

1.2 Why Maintain Trig Stations?

Under the Surveying and Spatial Information Act 2002 (NSW Legislation, 2015), by default, the Surveyor General is responsible for maintenance of survey marks:

> *9 Maintenance and repair of permanent survey marks*
>
> *(1) The Surveyor-General may, from time to time, cause notice to be given to any public authority of the location of any permanent survey marks that are located on land that is subject to the authority's control or management.*
>
> *(2) A public authority to which such a notice is given must ensure that all permanent survey marks identified in the notice are kept in good condition and repair.*
>
> *(3) On the application of a public authority to which such a notice is given, the Minister may direct that it is the duty of the Surveyor-General, and not the public authority, to keep any or all of the permanent survey marks concerned in good condition and repair.*

The Surveyor General may delegate maintenance of survey marks on public lands to the relevant public authority, but in practice this does not occur. Instead, LPI acts on behalf of the Surveyor General to carry out maintenance of survey marks, in cooperation with other public authorities where applicable.

Historically, trig stations were built and maintained by LPI staff (i.e. Piling Overseers) on an ongoing day-to-day basis. The last Piling Overseer retired towards the end of the 20th century and since that time trig stations have received minimal or no maintenance, except those close to regional offices or by special request on an ad-hoc basis. However, the ongoing

maintenance of trig stations is of particular importance due to their significant structures and placement in what are usually high-profile locations (e.g. lookouts and hilltops). Such marks in the public eye would reflect poorly on the surveying profession if they were to be indefinitely kept in a state of disrepair. Furthermore, it is critical to show landowners that the survey infrastructure located on their land is valued and maintained by LPI – this provides an incentive for landowners to identify with ‘their’ trig and maybe help look after it, as is sometimes done in the United Kingdom.

It is also important to maintain trig stations in order to keep LPI’s records contemporary. Regular visits will keep track of changes in land ownership, variations to access paths, such as fire trails or 4WD tracks and any new structures or improvements near the site. Up-to-date information is often required to help evaluate the importance of an existing trig station, which may be competing with proposed telecommunication infrastructure or other development. Furthermore, as indicated earlier, active maintenance practically demonstrates the State’s and authority’s continued interest in a site.

However, it is recognised that it is neither viable nor justifiable for the entire network of trig stations to be maintained in the modern era, particularly in light of the increasing number of active trig stations being installed as part of the ongoing expansion of CORSnet-NSW. Recalling that it took 100 years to cover almost half of NSW with a large number of passive trig stations by the 1960s, it is worth noting that it took only 5 years to cover more than two thirds of NSW with 150 active trig stations via CORSnet-NSW (Janssen, 2014), which are also much easier to maintain.

1.3 Technological Developments

Over the years, the rapid uptake of GNSS technology amongst the surveying profession has seen the perceived importance of passive trig stations wane as surveyors became less dependent upon line-of-sight to propagate datum. The establishment of CORSnet-NSW (Figure 3) has compounded this effect as new, more time-efficient positioning services such as Network Real Time Kinematic (NRTK) and Virtual RINEX are now available across most of the State (e.g. Janssen and Haasdyk, 2011; Janssen, 2013). Since CORSnet-NSW provides accurate, reliable and easy-to-use access to fundamental positioning infrastructure in real time, surveyors are generally no longer willing or desirous to visit passive trig stations to connect to datum.

LPI recognises that the trig station network is now mainly for its own internal use only. Trig stations can be located in remote areas and are often difficult and time-consuming to access and therefore have low appeal to external surveyors, when contrasted with the easy access and suitable quality of marks located along roads and urban corridors. Isolated trig stations are protected from road works and other development activities, which have destroyed many survey marks (PMs and SSMs) across NSW (e.g. de Belin, 2012; Ward, 2014), and as such are expected to have a perpetual lifespan. Thus, trig stations serve as the profession’s ‘insurance’ policy, and LPI guards the vital links in the chain to current and previous realisations and propagations of datums.

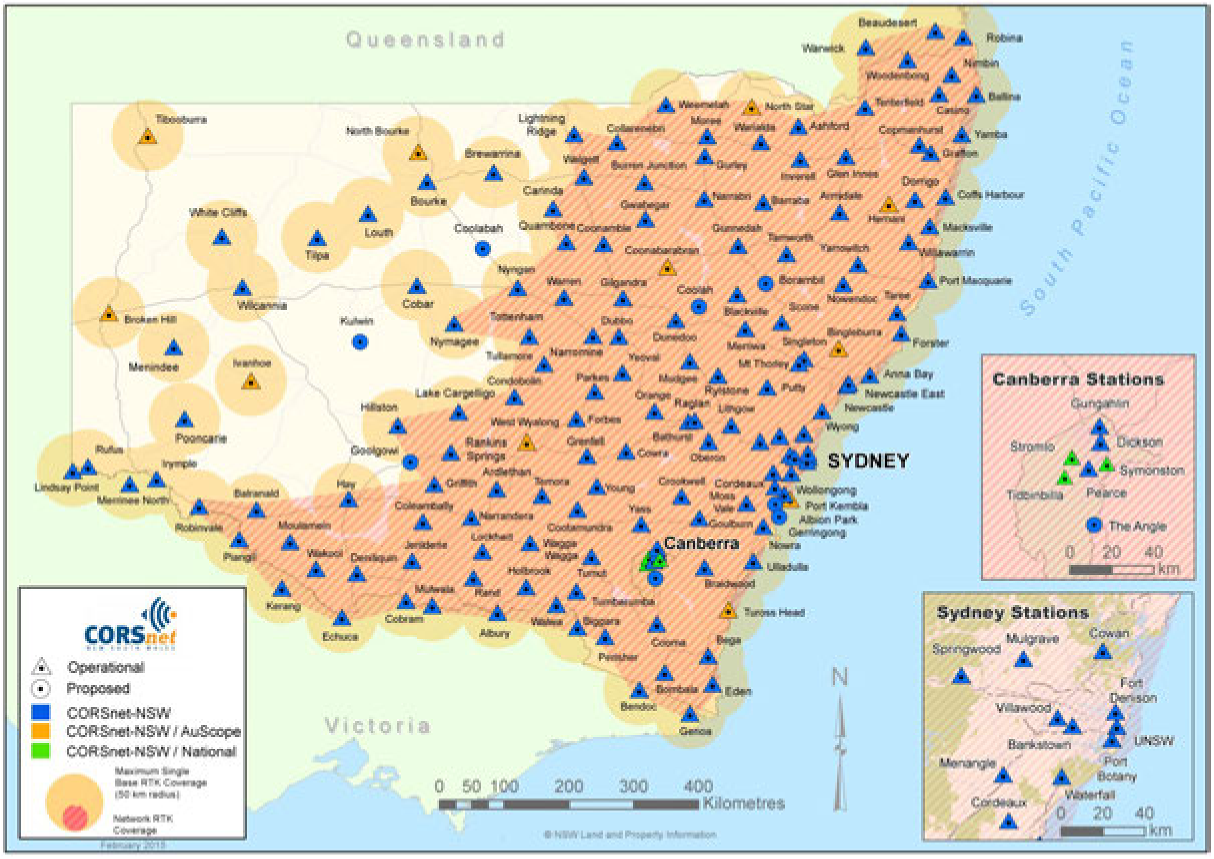

Figure 3: CORSnet-NSW coverage map as of February 2015 (LPI, 2015).

2 METHODOLOGY

2.1 Why Campaigns?

LPI has now adopted a campaign-style or project approach to carry out maintenance and upgrade of trig stations. This has originated from necessity as day-to-day survey operations place an increasing emphasis on doing 'more with less'. The tempo of lean, highly mobile operations is often rapid and often simple, but important things like trig maintenance and upgrade are placed low on the priority list and cannot be done on a best-effort 'while you are there' basis. Gone are the days of survey teams, often with many staff, carrying heavy bulky equipment to trigs, occupying them for multiple days as round after round of observations were made or there was time available while staff waited for the best observing conditions or times to occur.

Targeted campaign-style maintenance programs allow numerous trig stations in a geographic area to be visited, maintained and have their metadata records updated at a common epoch (generally during a 2-week time window). However, in today's prudent fiscal environment combined with technological advances, maintenance of the entire network of about 6,000 traditional trig stations in NSW is neither justifiable nor viable and therefore a targeted subset of the most important or desirable trig stations must be selected to direct preservation efforts towards.

2.2 TrigStar

Understandably, all trig stations are not equal. LPI's motivation to preserve and maintain trig stations can vary with regard to several criteria including:

- Previous survey work performed (e.g. number of terrestrial and GNSS observations).
- Prominence within the survey network.
- Ease of access.
- Suitability for GNSS observations.
- Suitability for further survey work (e.g. vegetation level, towers or structures, security).
- Historical significance (e.g. TS Kosciuszko, TS Cameron Corner and TS Barringun Zero Obelisk both on the NSW/QLD border).
- Local community identification (e.g. lookouts, public visits).
- Land ownership (e.g. trig reserve, state forest, national park, private land).
- Monument quality (e.g. survey pillar, trig plug and cairn, obelisk, GI nail in remains of wooden post).
- Condition of monument (e.g. decaying pillar, unpiled cairn, plug missing).
- Uniqueness of structure (e.g. concrete pillar on grand cairn, stone veneer vs. standard concrete pillar).
- Number of Deposited Plans connected to the trig station.

These criteria are used to assess each trig station across NSW and calculate a 'TrigStar' score out of 100 and a corresponding rating out of 5 stars (Figure 4). LPI has decided as a general rule to maintain the top 500-700 (i.e. about one in ten) passive trig stations with a rating of 4 or 5 stars. A similar process of rationalisation and upgrade was conducted in the 1970s and 1980s when trig stations not meeting certain standards in regards to permanence, capacity for occupation and usefulness for surveys were eliminated, while others were upgraded to concrete pillars (Rassaby, 1980).

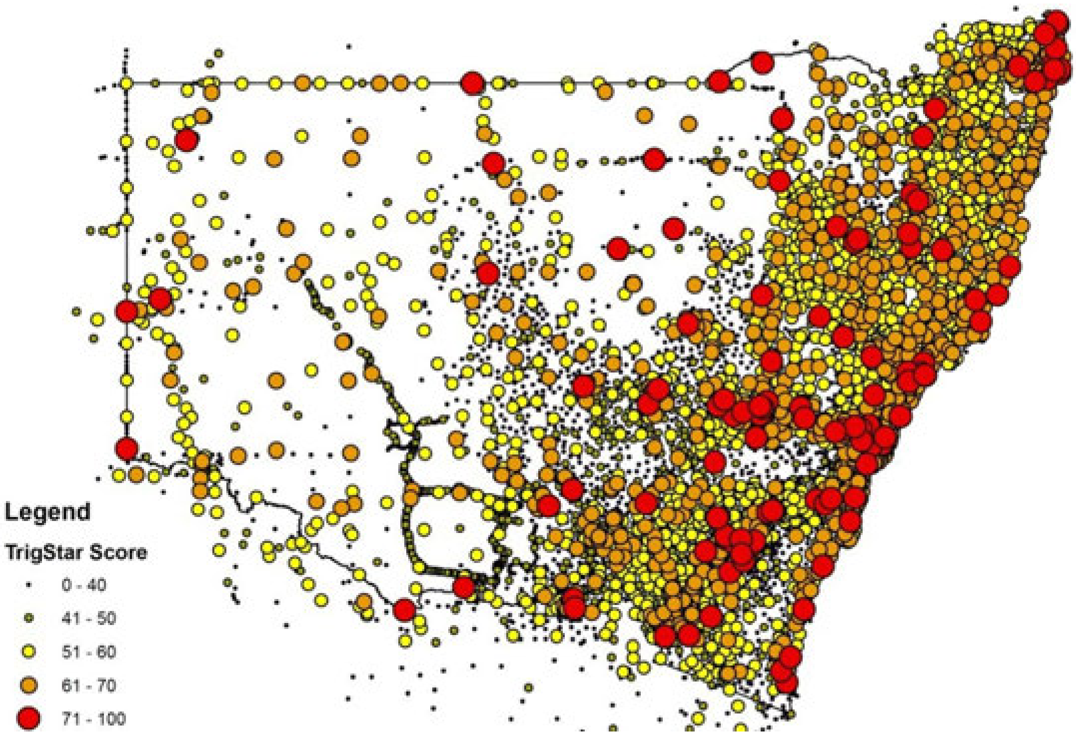

Figure 4: TrigStar scores across NSW (as of December 2014).

Initial TrigStar evaluations were conducted via an office desktop reconnaissance only, with records and imagery being searched to list potential candidates. In many instances, records were found wanting, missing, incomplete or completely out of date (i.e. 20+ years old). Generally, the trig stations with a score of 61 and above (i.e. a rating of 4 stars or higher) are selected as candidates for maintenance and upgrade in survey campaign planning. However, such a procedure failed to ensure a homogeneous coverage of suitable marks across the State. Hence additional candidates are often included, based mainly on geographical location.

In this context, it is worth noting that TS778 Lake George South Base (recall where the Trigonometrical Survey of New South Wales started all those years ago in 1867) is currently the highest-scoring trig station in NSW (Table 2). Obviously, the ranking is influenced by recent maintenance visits (all the listed trig stations but the destroyed have recently been visited) and therefore of a fluctuating nature. Currently about 115 trig stations are rated 5 stars, while about 870 trig stations occupy a 4-star rating. LPI aims to maintain approximately two thirds of all these trig stations.

Table 2: Current top-ranking trig stations in NSW according to TrigStar.

Rank	Trig Number	Trig Name	TrigStar Score
1	TS778	Lake George South Base	84
2	TS5816	Tarella	81
3	TS5566	Sutton Forest	80
4	TS3604	Observatory (destroyed)	79
5	TS2761	Kosciuszko	78
	TS5517	Mulley	78
	TS6424	Beelera	78
	TS6705	Cobar	78
	TS7065	Warral	78
	TS7273	Eden Breakwater	78
	TS7305	Burns	78
	TS7383	Westdale	78

2.3 Maintaining Geodetic Infrastructure

Keeping the State’s geodetic infrastructure current and ready for utilisation requires regular maintenance and upgrade. At LPI, this currently involves the following actions:

1. Completion of a new TrigStar form. Many of the initial TrigStar assessments were completed at the office desk rather than on site, which is prone to incomplete and inaccurate information.
2. Capture of new and fully digital photographic records. This allows for quick future use assessments with regards to condition of monument including eccentrics, suitability for GNSS survey, vegetation regrowth, etc. A historical record of the site at the epoch of the photograph is also captured.
3. Verification or update of landowner details and contact information. Changes in property ownership and contact details are noted and recorded for future use. Time invested in discussions with owners strengthens relationships with local hosts, helps reduce access issues for future users and helps promote an interest in (and hopefully a watchful eye on) the asset on their land.
4. Observation of at least 6 hours of dual-frequency GNSS data (preferably overnight).
5. Audit of eccentric marks.
6. Update of the access record. Many of the existing access details were recorded in the 1970s and are outdated. LPI continues with the traditional ‘chainage-and-event’ based access records but has added a new GPS Exchange Format (GPX) plot, based upon the

tracklog from a vehicular GNSS unit. This spatially referenced file can be imported into a Geographic Information System (GIS), overlaid on imagery and imported back into handheld GNSS units to aid navigation to the site.

7. Update of the visitation log. LPI's records show when the station was last visited, noting the purpose and any other major details arising from the visit.
8. Painting of the monument and repair if necessary. This is the aspect most visible to the public. Trig stations that are well maintained and without graffiti or damage demonstrate to landowners, hosts and the public that they are a valuable and important asset which is still in active use. Their condition also reflects on the image of the surveying profession.
9. Clearing of light regrowth vegetation. Maintaining trig stations includes keeping them in a condition that allows future GNSS observations. Light regrowth is therefore cleared to preserve the monument's usability.

Figure 5 shows a trig station before and after station maintenance was performed. Often the concrete base is also painted white to increase visibility in aerial images (and potentially allow the trig station to be used as ground control). Figure 6 illustrates a GPX access trail overlaid on aerial imagery, clearly demonstrating the benefit of generating visual access details. An example of the traditional visitation log and access entries available through SCIMS is shown in Figure 7.

It should be noted that not all trig stations are considered in these campaigns. GNSS CORS and reservoir trig stations are considered out of scope. LPI has a separate maintenance program for CORS, and reservoirs are now excluded due to WHS concerns and general poor monument quality related to mark movement caused by variation in water levels and seasons. In areas of low trig station density, silos were occasionally visited as a last resort where WHS requirements could be satisfied. Alternatively, selected PMs may be visited in lieu of low-scoring trig stations in order to allow a sufficient geometric spread of AUSPOS datasets (see section 2.4) across the area of interest.

Figure 5: TS1730 Cumber before and after trig maintenance.

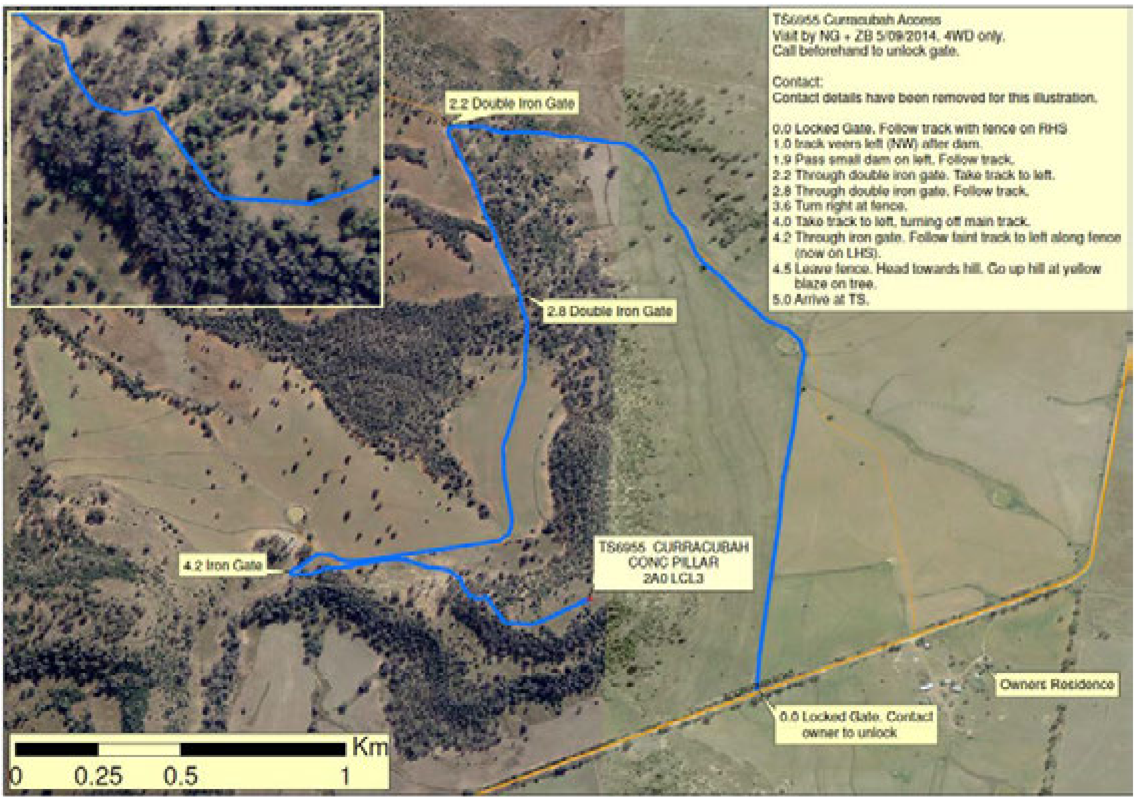

Figure 6: TS6955 Curracubah access including GPX trail overlaid on aerial imagery.

VISITATION LOG

Date	Organisation	Comments
1-NOV-2013	LAND & PROPERTY INFORMATION - BATHURST	STATION VISITED FOR MAINTENANCE AND GNSS SURVEY. GOOD VISIBILITY FOR GNSS. PILLAR, MAST AND VANES PAINTED
10-JUN-1992	LAND & PROPERTY INFORMATION - BATHURST	STATION VISITED FOR GPS SURVEY ON NUMEROUS OCCASIONS AROUND THIS DATE .
15-OCT-1980	n/a	NEW MAST AND VANES PLACED. NEW ACCESS.
2-JUN-1980	n/a	NEW MAST AND VANES PLACED PILLAR PAINTED.
20-DEC-1979	n/a	NEW MAST AND VANES PLACED
15-JUL-1976	DEPARTMENT OF MAIN ROADS	STEEL PILLAR FD.
29-JAN-1976	n/a	NEW MAST AND VANES PLACED.
19-NOV-1974	n/a	STEEL ROOF TOP PILLAR PLACED CONCENTRIC TO OLD TRIG PLUG. CANNOT CONFIRM CONCENTRICITY.
1-JAN-1969	n/a	MAST AND VANES PARTLY GONE CAIRN INTACT.
1-JAN-1892	DEPARTMENT OF LANDS	STATION ESTABLISHED. PLUG IN ROCK MAST AND VANES, STONE CAIRN.

ACCESS

Date	Vehicle Access	Walk Time (min)
1-NOV-2013	NONE	60

Description

Allow 20 minutes (drive) plus 60 minutes (walk).
0.0 From WOODBURN ROAD, turn onto CLYDE RIDGE ROAD at sign ?Pigeon House Mountain 14 km?.
1.6 Pass THREE FALLS ROAD on LHS. Pass sign ?Flat Rock State Forest?.
5.9 Pass McMAHONS ROAD on LHS.
7.7 Turn right onto YADBORO ROAD at sign ?Pigeon House Mountain, Yadboro?.
9.3 Pass sign ?Yadboro State Forest?.
12.0 Turn right onto PIGEON HOUSE ROAD at sign ?Morton National Park?.
13.2 Leave vehicle at car park and follow steep walking track to TS (60 minute walk).
16.3 Arrive at TS.

Note also that CLYDE RIDGE ROAD offers good access from other Trigs to the South of Pigeon House.

Figure 7: SCIMS visitation log and access for TS3737 Pigeon House (‘?’ indicates quotation marks in the original document, caused by computer interface glitches).

2.4 Improving Geodetic Infrastructure via AUSPOS and a Next-Generation Datum

Monument condition and metadata are not the only features that stale with age. Of paramount importance in a geodetic survey network are modern (i.e. at least dual-constellation), time-stamped, long-session GNSS observations. This enables time series analysis of mark movement and the ability to reprocess old raw data as algorithms improve. Such results may be later displayed on the web, much like currently done for CORSnet-NSW stations. As part of the next-generation datum project, LPI is currently retrieving up to 20-year-old GPS data sessions of 3 hours or more for reprocessing using modern tools.

It is well known that systematic distortions of up to 0.2 m horizontally and 0.3 m vertically exist in NSW between the legal coordinate system as realised by SCIMS, i.e. GDA94(1997), and observations in the more homogenous GDA94(2010) realisation of the national datum as provided by CORSnet-NSW and AUSPOS (e.g. Haasdyk et al., 2010; Janssen and McElroy, 2010; Gowans and Grinter, 2013). As an example, Figure 8 illustrates these distortions across the Central West. However, the exact extent of such distortions is not fully known. Test points are required throughout the State, with higher densities in the most affected areas, and trig stations provide the best standpoint for such observations, now and into the future.

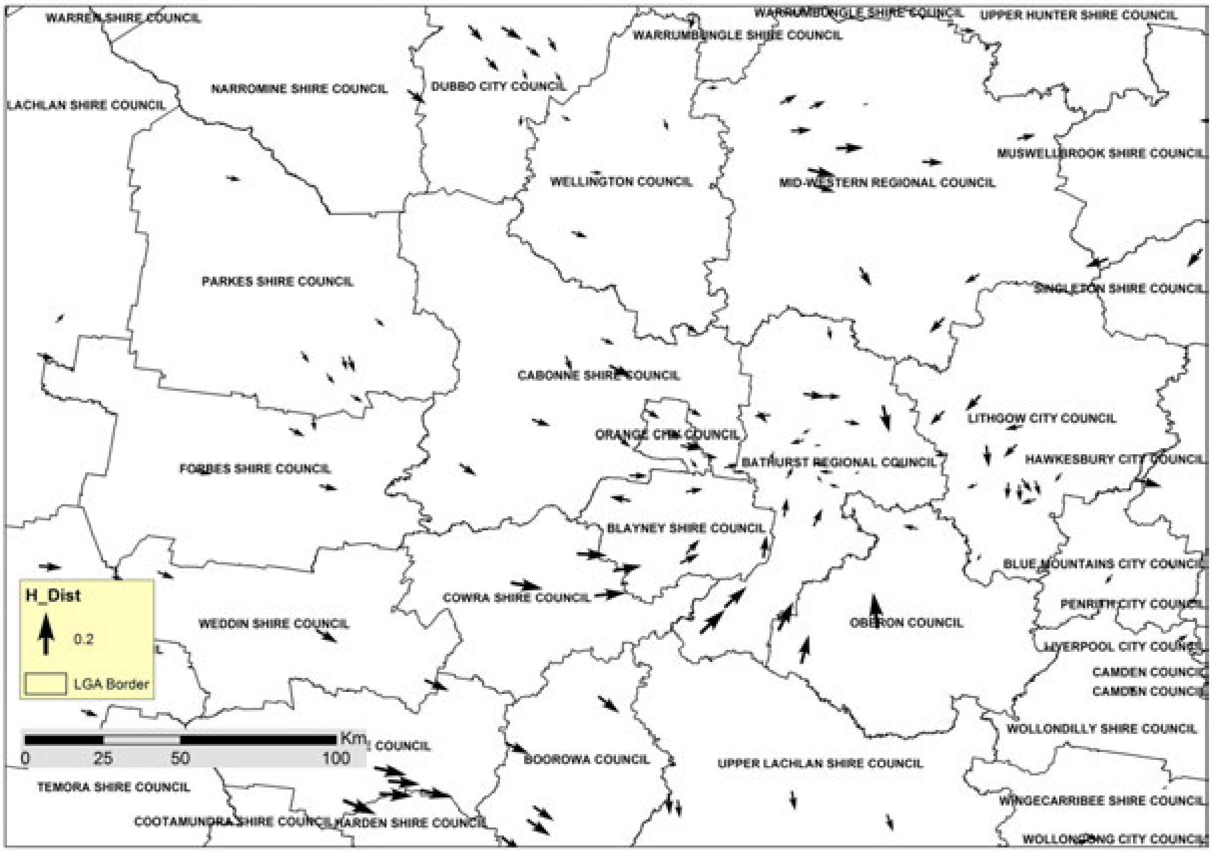

Figure 8: GDA94(1997) to GDA94(2010) distortion vectors across the Central West.

Removing these distortions between local control in GDA94(1997) and coordinates in GDA94(2010) requires a re-adjustment of the entire NSW network, without a hierarchy of fixed control, which will occur as part of the national adjustment to produce a next-generation datum for Australia (Haasdyk et al., 2014b). In the interim, a site transformation is required to relate CORS-derived positions to the local (and legally accepted) ground control available

in NSW via SCIMS (Haasdyk and Janssen, 2012). Trig stations are the logical sites to occupy and measure distortions, as this is where the original distortion originated in the first place.

In this context, it should be noted that LPI also carries out local tie surveys to connect each CORSnet-NSW station to the surrounding ground survey control (Gowans and Grinter, 2013), of which connections to passive trig stations typically form more than half of the overall survey effort. The immediate goal of each tie survey is to propagate the local distortions in GDA94(1997) and AHD71 to the CORSnet-NSW station, producing a best 'local-fit' position. However, the ultimate goal is the opposite, i.e. re-adjusting the entire state survey control network and propagating the Regulation 13 CORSnet-NSW station coordinates (GA, 2015c) outward to the ground survey network, via passive trig stations, as part of the next-generation Australian datum.

As part of the aforementioned list of actions completed during a trig station maintenance visit (see section 2.3), extended GNSS observations (in excess of 6 hours, often overnight and up to 24 hours) are captured to be utilised in the next-generation datum adjustment. These datasets are processed via AUSPOS (GA, 2015b) for validation and internal analysis before being submitted for inclusion in the national survey network adjustment to form the new national datum. These observations form a significant part of a greater effort currently underway by LPI towards datum improvement (Haasdyk et al., 2014a).

With the densification of CORSnet-NSW and its inclusion in AUSPOS processing, LPI is moving towards position-based results becoming increasingly utilised in place of the traditional survey network composed of GNSS baselines. These positions will constrain the traditional GNSS baselines in the next-generation datum adjustment, providing a homogeneous datum realisation by removing existing distortions. Figure 9 illustrates LPI's current database of AUSPOS datasets across NSW.

The benefit of constrained centring offered by concrete or steel pillared trig stations helps minimise or remove some errors, however small. This should in turn help authorities achieve their ambitious goal of achieving an Australian datum realisation of better than 20 mm.

Often remote trig stations only have AHD71 heights derived from trigonometrical heighting, which is accurate to about 0.5 m. The occupation of such stations by GNSS and derivation of AHD71 heights from ellipsoidal heights and AUSGeoid09 (GA, 2015a) has the potential to produce values far better than those that currently exist, provided that the limitations in this methodology are remembered.

Long GNSS sessions, at homogenous points distributed across the State, will also help ensure that suitable transformation grids and tools (e.g. National Transformation Version 2 grids) can be developed, which will have to be used initially for transforming up to about 90% of marks in SCIMS to the next-generation datum.

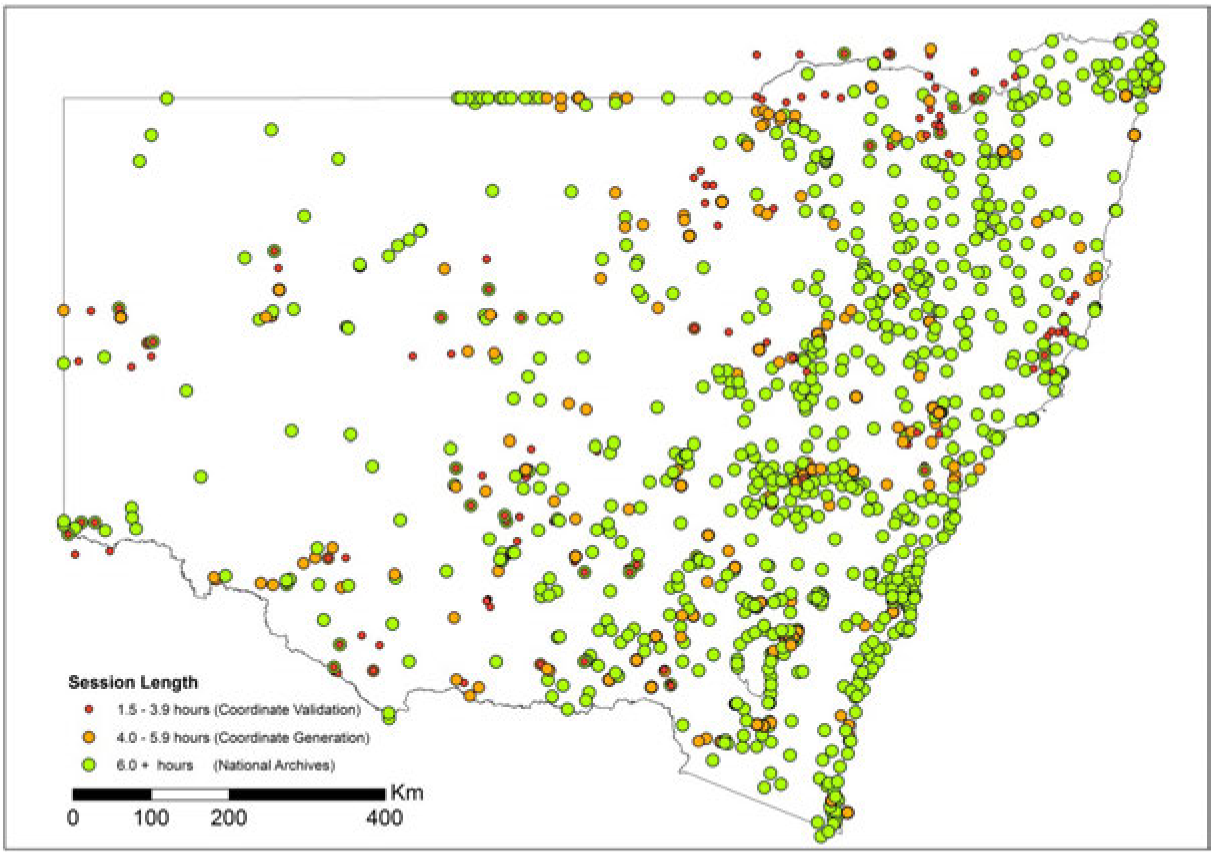

Figure 9: LPI's current AUSPOS dataset collection (as of January 2015).

2.5 Work Practices

Resources and time are limited and require careful planning to successfully complete a trig maintenance campaign. Once the campaign's sites have been selected based on TrigStar rating and geographical distribution, those chosen are divided into Local Government Areas (LGAs) and allocated to individual LPI survey teams. Each LGA is then broken into daily work plans, and each day is typically split into two parts. The first part involves revisiting sites from the previous day to collect the GNSS receivers that observed overnight, while the second consists of visiting and upgrading new sites. From experience, it is generally feasible for one survey crew to visit and occupy six sites in a standard 10-hour field day.

Considerations must be made with regards to distance between sites, organising access with landowners, keeping batteries charged, difficulty of access, technical 4WD skills required and most importantly getting home safely at the end of each day.

Work Health and Safety (WHS) considerations must come before all others – Safety First is LPI's motto. Related practices ensuring a safe working environment include:

- Two-person field parties operating in 'buddy' mode for safety.
- Remote and/or senior first aid training for staff.
- 4WD vehicles equipped with long-range tanks, driving lights, HF radios, spare tyres and recovery gear.
- 'Grab bags' for sites that require a walk to.
- Call-in and call-out procedures to notify the project manager of start and finish times and expected work to be completed each day.

- EPIRB (Emergency Position Indicating Radio Beacon) devices for use in emergency situations.
- Safe Work Method Statements (SWMS) and risk assessments.

3 SUMMARY OF RESULTS AND LESSONS LEARNED

LPI has so far completed four campaigns utilising this approach towards trig station maintenance in NSW (Figure 10). Campaign 1 was a pilot focussing on the Central West (May 2013), campaign 2 concentrated on the South Coast and ACT border (October 2013), campaign 3 focussed on the Mid North (September 2014), and campaign 4 concentrated on Southern NSW (February 2015).

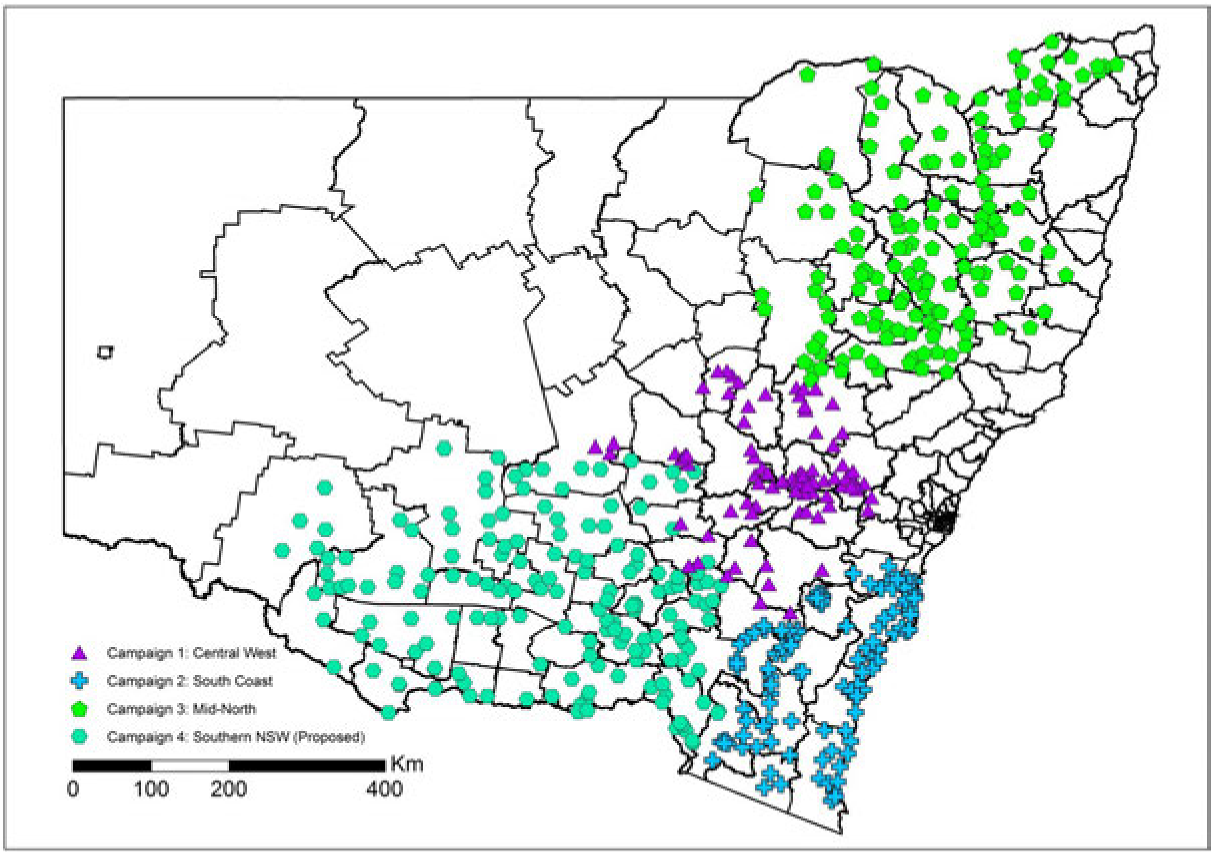

Figure 10: AUSPOS datasets gathered from trig maintenance campaigns in NSW to date.

Relevant metadata for each campaign conducted to date is summarised in Table 3. The significant amount of deliverables achieved with these targeted survey campaigns is clearly evident from the data shown. The campaigns have proven to be a very efficient and effective way to maintain and upgrade the State's geodetic infrastructure and collect valuable data for the adjustment of the next-generation Australian datum. After three campaigns, 350 trig stations have been maintained in this way, with a fourth campaign accounting for an additional 130 or so sites.

Table 3: Trig maintenance campaign details to date (proposed values for campaign 4 shown).

Campaign	1	2	3	4
Area of Operation	Central West	South Coast	Mid-North	Southern NSW
Epoch	May 2013	October 2013	September 2014	February 2015
AUSPOS Sessions	80	120	150	140
LGAs covered	13	12	16	31
Crews	3	5	6	5
GNSS receivers	18	25	25	25

The outlined process has been steadily refined and improved from one campaign to another. For example, following feedback obtained from field staff during the first campaign, the TrigStar assessment form was improved to take into account additional factors such as site security and presence of other structures (and RF interference) at the site. In later campaigns, the concrete base of trig pillars was often painted for use as aerial mapping targets, noting the height difference from the pillar plate to the concrete base. Some campaigns required an additional week of 'mopping-up' work for a team to visit stations that were initially missed due to inclement weather, time constraints or other unforseen delays that occur in the field.

4 FUTURE PLANS

It is anticipated that in the spirit of ongoing maintenance and upgrade, LPI will repeat one campaign annually (with a complete cycle taking approximately 5-7 years) in an effort to keep a contemporary set of GNSS observations and records for two thirds of its 4-5 star trigs. Sydney and regional LPI survey offices will continue to maintain and upgrade similar trig stations via day trips on an 'as needs' basis, of no less than a 5-7 year cycle.

Following the significant efforts undertaken towards trig station preservation and upgrade, and the next-generation Australian datum, LPI proposes a similar campaign-style project, called "Saving AHD", to maintain and upgrade the State's fundamental levelling infrastructure. It is anticipated that some 600-800 original 1970s Australian National Levelling Network (ANLN) marks (i.e. a minimum of three marks per level run) will be preserved and upgraded to meet current and future requirements.

5 CONCLUDING REMARKS

On behalf of the Surveyor General, LPI has a legislative, regulative responsibility to maintain the geodetic control network in NSW. As such, LPI is the custodian of some 250,000 marks in SCIMS, which includes about 6,000 traditional 'passive' trig stations as well as more than 160 'active' GNSS CORS. Keeping the geodetic component of the survey control network current and ready for utilisation requires regular maintenance and upgrade. Considering that it is neither feasible nor justifiable to maintain all passive trig stations across NSW in the modern era, particularly in light of the continuing expansion and increasing use of CORSnet-NSW, LPI has introduced a campaign-style trig station rationalisation, maintenance and upgrade program to maintain about 500-700 trig stations of high significance.

This paper has outlined an effective strategy to preserve geodetic infrastructure across NSW, connect traditional survey infrastructure with modern-day satellite-based positioning technology and contribute new GNSS observations towards the next-generation Australian datum. The results of conducting four such campaigns have proven that this method is able to

efficiently maintain the physical trig station infrastructure and associated metadata while capturing new, time-stamped, high-accuracy GNSS observations. These improvements will not only have a significant and lasting effect in regards to the next-generation Australian datum but also help increase the general public's appreciation of the surveying profession in general and the importance of survey infrastructure and its preservation in particular.

ACKNOWLEDGEMENTS

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A Practical Assessment of GNSS Angular Suitability for Cadastral Surveys

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ABSTRACT

This is a follow-on paper to "Looking at Cadastral GNSS from a New Angle", which was presented at APAS2014 and identified that the current Surveying and Spatial Information Regulation 2012 does not cover the angular component of radiated measurements regardless of the technology used to measure them. This paper investigates Global Navigation Satellite System (GNSS) angular observations to determine how they relate to traditional angular observations. In order to achieve this, a 3" total station supplies direct comparisons to static GNSS observations measured on the same day over three triangular networks of varying lengths. From this comparison, options are considered regarding the suitability of GNSS for cadastral surveying from an angular perspective in relation to the current Regulation. Like all things cadastral, there are no easy, straight-forward answers – and this is no different. The usual head-to-head comparison will always be won by the total station when considering distances less than 500 m. What is interesting is that when the practical application of cadastral surveying is added into the equation, then there are some real possibilities to use GNSS over short distances and still obtain the same result as a total station. It is a matter of the correct application in an appropriate situation.

KEYWORDS: *Cadastral surveying, GNSS, Surveying & Spatial Information Regulation 2012, angular tolerance*.

1 INTRODUCTION

Have you ever considered how well Global Navigation Satellite System (GNSS) technology can measure an angle in comparison to traditional methods of survey? This question has very broad implications and may assist in determining where GNSS may be used for cadastral applications. The ultimate question of "What is an acceptable azimuth swing across a cadastral survey?" would define the end goal. However, "How long is a piece of string?" is an easier question to answer. Cadastral surveys are usually small by comparison to geodetic surveys, yet there are occasions when the ability to link to small traditional surveys together with a great distance between them is the key to solving a problem.

This paper looks at that problem, i.e. "How to use GNSS to link traditional surveys together?" The critical point in this process is "How close can the GNSS stations be at each end of the network and still maintain cadastral accuracies?" or "How well can GNSS technology measure an angle in comparison to traditional methods of survey?" In order to make the comparison, three simple triangular networks were used to make traditional angular measurements, followed by GNSS measurements taken on the same day. This allows direct comparison between the derived angles for each method.

2 PRACTICAL EXAMPLES OF THE PROBLEM

The first project example is located in Woods Reef, north-west of Gunnedah, NSW (Figure 1). This is an unsurveyed parcel, approximately 4 ha (10 ac) in size (i.e. 135 m x 300 m), gazetted in 1911 and bounded by a 2,000 m boundary on the west, Portion 77 on the north and east last surveyed 1917 and a creek to the south. The terrain is steep to mountainous. There are two Reference Mark (RM) trees 25 m apart in the north-west corner and another tree 2,000 m away to the south. After a 3-hour walk, just to see if the southern tree was there, it was decided that GNSS was the best option to use to survey the long line. However, how far apart do the stations at each end need to be to avoid an azimuth swing in the survey?

The survey process was to place a traverse along the road in the north and place a station near the southern tree with a reference station also in the south for traditional observations to radiate all the necessary cadastral objects. This forms a braced quadrilateral to base the survey on.

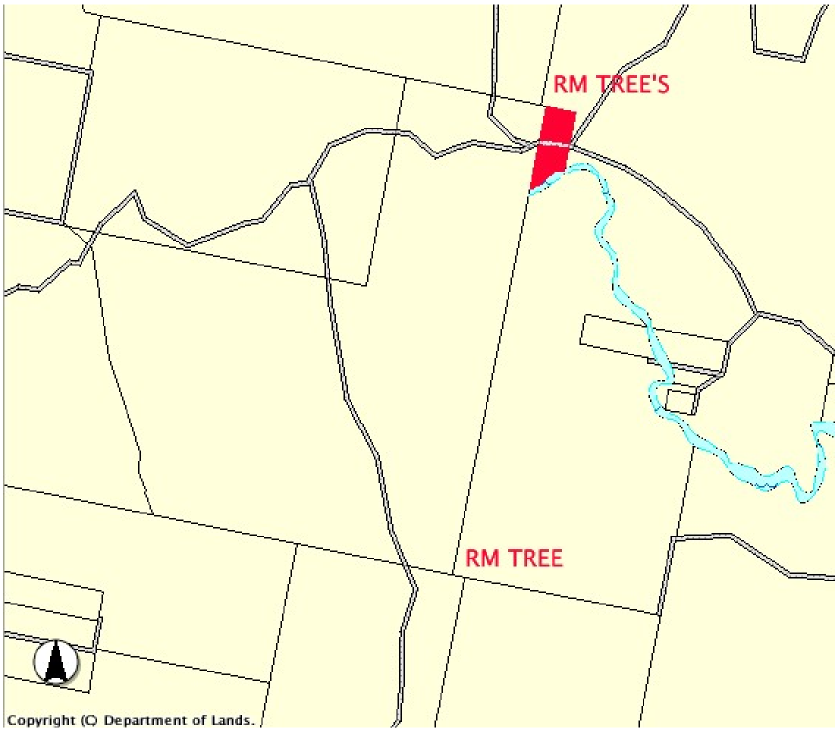

Figure 1: Woods Reef.

The second project example is located in Thalgarrah, north-east of Armidale, NSW (Figure 2). This is an unsurveyed parcel, approximately 10 ha (25 ac) in size (200 m x 400 m), gazetted in 1880 and bounded on the west and north by Portion 201 surveyed in 1880 and the creek on the east and the road to the south. After searching 4 km^2 and only finding three RM trees (one 1,200 m north-west and two 2,000 m south-west of that one), GNSS was decided to link the trees and to put in the necessary Permanent Marks (PMs) for the survey.

The survey process was to create two triangles, one in the north and one in the south. This created a braced quadrilateral with two triangles at the ends to link the trees and then a GNSS PM spine along the road braced by the surrounding trigonometrical station network.

Figure 2: Thalgarrah.

The common thread here is the short distances at the ends of the braced quadrilaterals. The long sides are very well suited to GNSS technology. However, how well are the short sides suited to being used for a cadastral survey with the accuracy determined by GNSS? Will the errors within the GNSS measurements over short lines introduce an azimuth swing between the two ends of the survey?

Figure 3 provides an exaggerated example of azimuth swing. The white and yellow lines show what we believe is correct, i.e. the results from the instrument. In this example, to make it clear and easy to follow (this would rarely happen in the real world and would not be done with intention), the yellow lines are parallel. The circles are the error circles, which are inevitable and unavoidable. The red lines are the actual result. While we would believe that the two short red lines are parallel, they are in fact not but we are unable to determine this. In practice, it is better to have a triangle at each end to have an angular check at each end. For simplicity of the concept this is not shown.

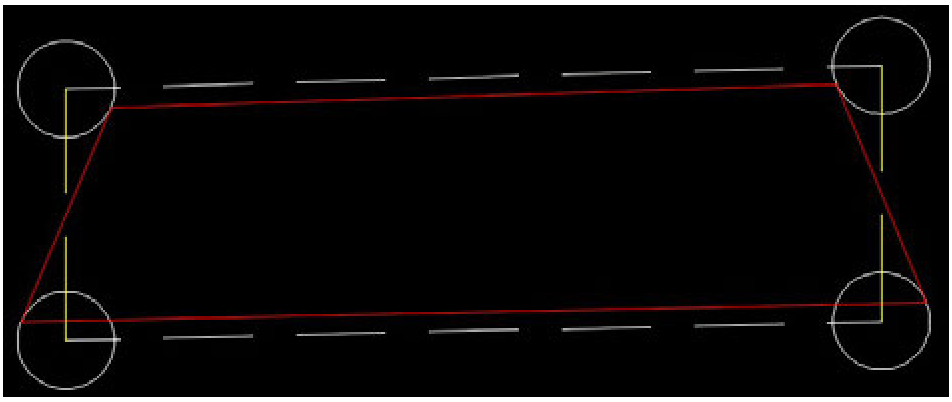

Figure 3: Exaggerated example of azimuth swing concept.

This is common in rural surveys when longer distances are needed to be travelled but it is not possible to survey the actual point as it is under tree cover or there is some reason that it is not possible to set up on the survey mark. Maybe there are a few points of interest, so it is better to use a total station to gather all the information in the local area and then move to the next corner where the process could be repeated.

The minimum distance between the stations needs to be determined with regard to the quality of the survey being undertaken or with regard to the current standard of a total station's ability to measure angles. This is to ensure an azimuth swing is not introduced (this would affect the traverse), which will need to comply with the Surveying and Spatial Information Regulation 2012 (NSW Legislation, 2014), therefore allowing GNSS to be use in cadastral applications with the knowledge that if traditional methods are used a similar answer will be achieved.

3 CURRENT TOTAL STATION ABILITIES

Current total stations are rated to the ISO12857 standard, which is stated at 95% (Zeiske, 2001). Therefore, as an example, a total station rated at 3 seconds of arc (3") should be able determine the mean of each set, i.e. Face Left / Face Right (FL/FR) pair, within ±3" of the mean of all the sets, 95% of the time. In order to test this claim, 100 rounds were read with a 3" instrument using Automatic Target Recognition (ATR) using the field test layout described in section 4.

From Figure 4, the results make obvious that modern instruments need to use traditional best practice methods to turn repeatable angles. There is a belief that modern instruments read both sides of the circle for any reading. This may or may not be true – the results show that even if they do read both sides of the circle for every reading, there are other errors that need best practice when turning angles to achieve reliable results. Simply turning FL is not good enough to achieve the manufacturer's specification. Also, it is good practice to read both faces to check pointing error and average to wobble in the prism assuming it is a pole shot to a feature. The point to note from the graph is that a single FL/FR mean angle from a 3" instrument may fall ±9" at 95% either side of the true mean of the angle.

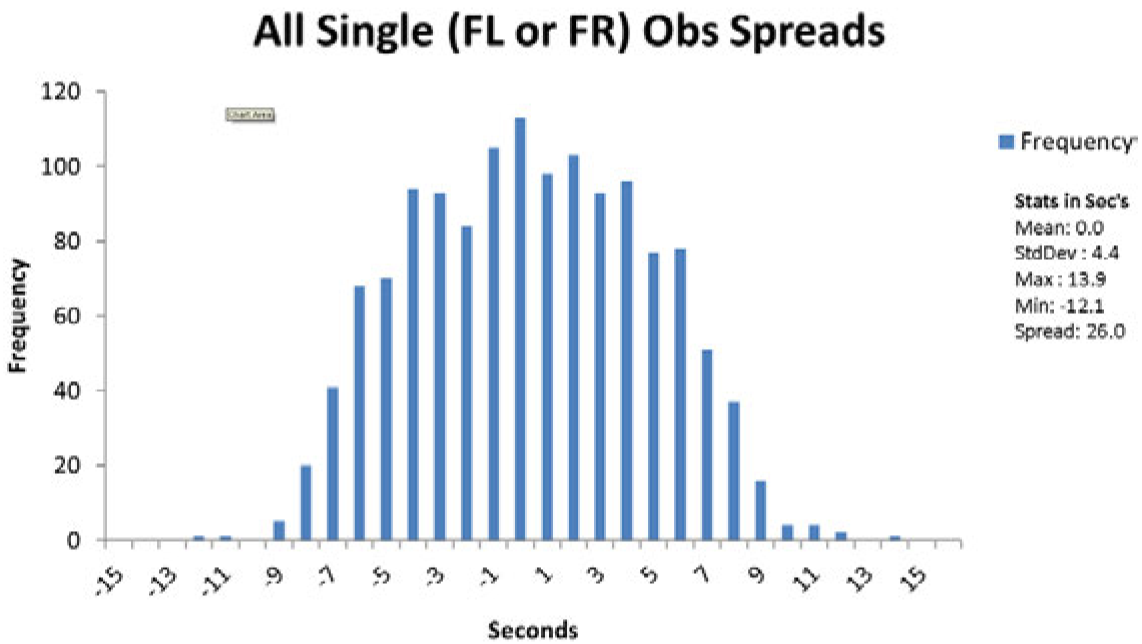

Figure 4: Spread of single observation.

Best practice is to turn rounds. It is suggested that four rounds, i.e. halving the error in the instrument (Fenwick, 2014) should be used, as with a modern instrument it takes little time to achieve good results. Surprisingly, from Figure 5, turning a round of four sets still only gets within ±2" of the actual true mean angle that would result from 100 rounds. The other

observation is, by using the same data for the 100 rounds and splitting it into sets of four, as would be the practice in the field, the standard deviation drops from 2” to 1”. This leads into the question of what to keep and what to throw away in the field. It is not possible to take a large amount of readings, take it all back to the office and process it while doing a cadastral survey. While processing this data, the rule of thumb that was used is if it is outside the manufacturer’s specification, discard the data and re-read, i.e. in this case any mean larger than 3” from the mean of four sets was discarded. All of these conclusions are very interesting on their own. However, what does this mean in terms of how GNSS angles and total station angles compare?

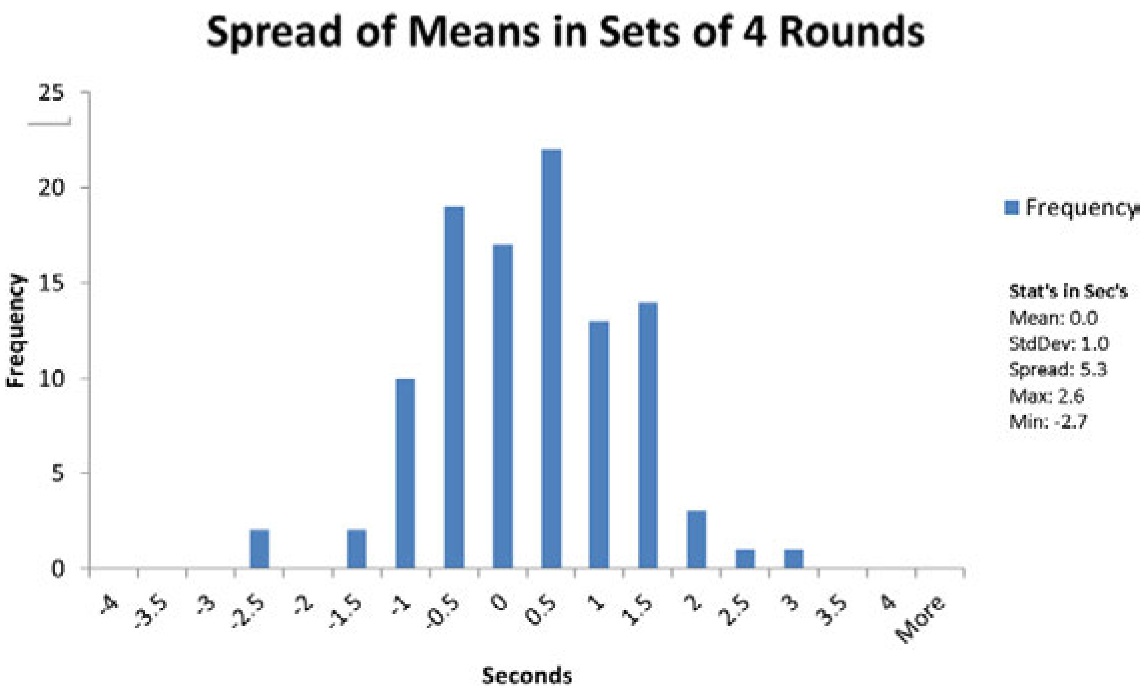

Figure 5: Spread of means in set of 4.

There are two schools of thought used when surveying. The first is to do it fast, do the bare minimum to get the job done and get it close most of the time. Just turning one FL/FR pair falls into this category. The other is to spend a little extra time, have some redundancy to get a better result and know it is right. This involves turning rounds and assessing the sets to ensure they fit within the specification. There is a third option usually reserved for topographical work where it is not practical to mean all the observations, known as the single FL observation. Out of interest, why not compare all the options to the GNSS results.

Table 1 shows the expected abilities of a 3” total station as defined by reading 100 rounds with this instrument. It needs to be pointed out that this is just the error associated with the instrument, not including the setup error or the target location error. The total error would need to include all of the errors in the measurement. This will be dealt with later in the paper.

Table 1: Abilities of a 3” total station.

Distance (m)	2” 4 rounds (±mm)	4” single FL/FR (±mm)	9” single FL (±mm)
150	1.5	3.0	6.8
250	2.5	5.0	11.3
400	4.0	8.0	18.0
550	5.5	11.0	24.8

Table 2 relates a physical object used in cadastral surveying to the error circle. This allows a practical appreciation of the errors being discussed to be considered. An error of ±10 mm sounds big, however looking at a Galvanised Iron Nail (GIN) in a fence it really does not seem that bad.

Table 2: Practical perspective of errors.

Item	Approx. Size (Diameter)	Error Association (Radius)
Concrete Nail Head	4 mm	±2 mm
Clout Head	9 mm	±5 mm
GIN	19 mm	±10 mm
Road Spike	40 mm	±20 mm

4 FIELD TEST LAYOUT

The field test site was chosen to allow three triangles to be laid out with two sides at approximately 150 m, 250 m and 400 m to simulate real-world survey situations. It becomes very difficult to find lines of sight more than 500 m long so that would then involve traversing between the marks with intermediate stations. In order to remove the movement variable, the marks were placed and then measured by both methods on the same day or consecutive days as would happen in a conventional survey. This then allows direct comparison between the angles determined by each method on the same day. As the same marks were used over the course of the field trial, some comparisons could be done but this will include any ground movement. This would not be any more than if the same thing occurred in a normal survey and is a good indication of achievable survey accuracy. In Figure 6, station 1001 is the vertex, stations 1004 and 1012 are at 150 m, stations 1007 and 1013 are at 250 m, stations 1002 and 1008 are at 400 m, and station 1003 is at 550 m.

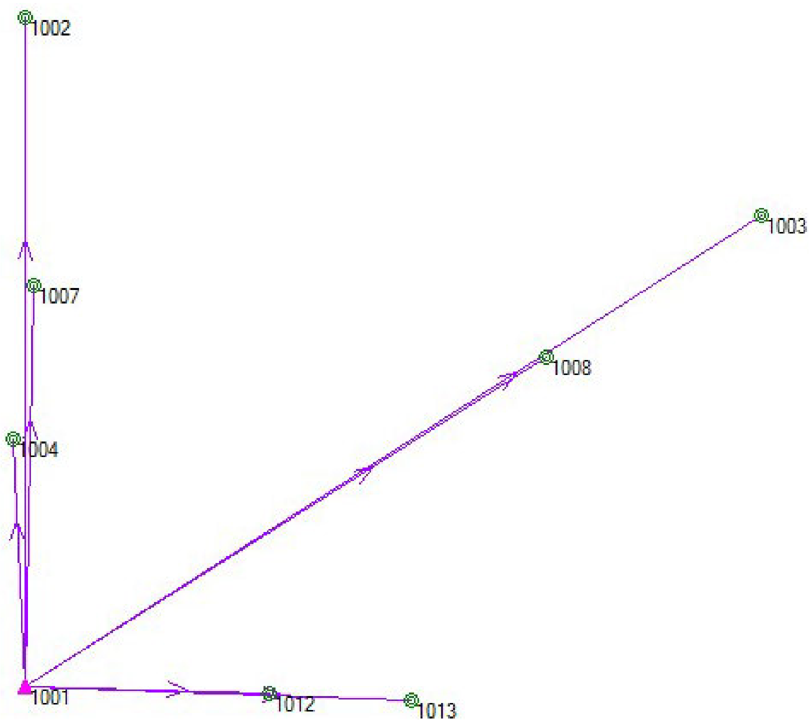

Figure 6: Field test layout.

5 PROCESSING

In order to compare like with like, the raw unadjusted data is compared. This allows a direct repeatability comparison between the two instruments. As each day has both total station and GNSS data, there will be no bias based on setup errors. The aim is to determine the error circle expected for an average setup in normal working conditions, not some unusable error solely based on instrument specification as discussed earlier.

5.1 Total Station (TPS)

From the 100 angles read in sets of 20, the mean FL/FR pairs were sorted by point number and coordinates calculated using horizontal distance. Using this information, the means and standard deviations were calculated for each point. This then allowed the differences in Easting and Northing to be determined along with the error circle for each mean. The error circles were compared to the mean ±3 times the standard deviation to clean the data of outliers as all cadastral work is determined at the 95% confidence level. The data for the other days only contained enough sets to be ‘happy’ in the field that there was agreement between the sets, i.e. there is not more than 3” between the means of each set. This is standard operating procedure when reading rounds on a 3” instrument.

5.2 GNSS

After downloading the data, the processing follows the standard methodology of checking antenna heights and cleaning the station names. Lines are then processed keeping the vertex fixed as the ‘known point’. Its coordinates were determined by using the first day’s data from station 1001 (the vertex) with three Continuously Operating Reference Station (CORS) sites belonging to CORSnet-NSW (e.g. Janssen et al., 2013; LPI, 2015): Singleton, Newcastle and Bingleburra CORS. The actual value chosen is immaterial as it is now the ‘known point’ that all values are determined from. This was only done to allow GNSS processing to occur as the starting coordinates need to be realistic to get good answers.

The processing was undertaken using Leica Geo Office (LGO), each line was processed and the MO value and the horizontal and vertical indicators recorded. The report data was then viewed to look for cycle slips to try to improve the result. The new MO and indicators were compared and the best option selected.

5.3 Comparison Methodology

The aim is to compare the total station (TPS) to GNSS in the field. To this end, the processing has held the vertex fixed and coordinates have been produced for all the other stations by both TPS and GNSS. At each station the mean coordinates for both TPS and GNSS were determined and then the difference between each observation was compared to its respective mean. This was achieved by comparing Eastings (E) and Northings (N) to the mean Eastings and Northings, resulting in coordinate differences (dE and dN). In other words, to avoid misunderstanding, each TPS observation was compared to the TPS mean and each GNSS observation was compared to the GNSS mean. After the differences were derived, a distance to the mean was determined using Pythagoras’s Theorem. These calculations resulted in the ability to determine the error circle for each observation at each station and provided the standard deviation to the mean of the observations.

Similarly for an angular comparison, each angle was determined by both TPS and GNSS for each session and these were then used to determine the mean of each angle. Each of the observations was then compared to the mean to produce a difference and this produced a standard deviation for both the TPS and the GNSS. The angle for the GNSS was determined by subtracting one direction from the other direction to calculate the angle, e.g. direction A to B was subtracted from A to C to determine the angle BAC. This was repeated for the 150 m, 250 m, 400 m and 550 m triangles.

6 RESULTS

6.1 TPS Instrument Errors

The TPS instrument errors were determined from turning 100 rounds using ATR and comparing each resulting coordinate from each mean FL/FR observation to the mean of all observations or grouped in sets of 4 and sets of 20 to each station. All were read on the same setup and are therefore free of setup errors (Table 3).

Table 3: TPS instrument coordinate errors in millimetres (95%).

Station	Single FL/FR	Distance (m)	Sets of 4 Mean	Sets of 20 Mean
1004	3	150	3	0
1012	3	150	3	1
1007	5	250	4	1
1013	4	250	4	1
1002	2	400	0	2
1008	6	400	6	2
1003	9	550	9	2

6.2 TPS Coordinate Comparison

Observations were read over several days compared to the mean of all observations (each observation being the mean of a set of 4) to each station by comparing the coordinate values, hence including setup errors (Table 4).

Table 4: TPS coordinate results in millimetres.

Station	Mean	Std Dev	Distance (m)	95% Error
1004	2	1	150	4
1012	3	1	150	6
1007	2	1	250	4
1013	1	0	250	2
1002	2	1	400	3
1008	2	1	400	3
1003	4	1	550	6

6.3 TPS Angular Comparison

Observations were read over several days compared to the mean of all observations to each station by comparing the angular values, hence including setup errors (Table 5).

Table 5: TPS angular results in seconds.

Station	Mean	Std Dev	Distance (m)	95% Error
1004	0.000	2.2	150	4
1012	0.000	2.5	150	5
1007	0.000	0.8	250	2
1013	0.000	2.2	250	4
1008	0.000	2.6	400	5
1003	0.000	2.6	550	5

6.4 GNSS Coordinate Comparison

Observations were read over several days compared to the mean of all observations to each station by comparing the coordinate values, hence including setup errors (Table 6).

Table 6: GNSS coordinate results in millimetres.

Station	Mean	Std Dev	Distance (m)	95% Error
1004	5	3	150	10
1012	4	3	150	10
1007	3	1	250	5
1013	5	3	250	11
1002	3	1	400	5
1008	8	4	400	15
1003	5	3	550	12

6.5 GNSS Angular Comparison

Observations were read over several days compared to the mean of all observations to each station by comparing the angular values, hence including setup errors (Table 7).

Table 7: GNSS angular results in seconds.

Station	Mean	Std Dev	Distance (m)	95% Error
1004	0	2.6	150	5
1012	0	6.5	150	13
1007	0	1.9	250	4
1013	0	4.2	250	8
1008	0	0.8	400	2
1003	0	1.6	550	3

6.6 TPS to GNSS Angular Comparison

Observations were read over several days comparing the angle determined by subtracting one direction from the other direction by TPS and then by GNSS read on the same day on the same setup. The difference is reported as TPS angle minus GNSS angle. The result of each day is then compared to all results and is therefore free of setup errors (Table 8).

Table 8: TPS to GNSS angular results in seconds.

Station	Mean	Std Dev	Distance (m)	95% Error
1004 - 1012	2.8	4.9	150	12.6
1007 - 1013	2.0	7.0	250	16.0
1002 - 1008	2.1	3.6	400	9.3
1002 - 1003	1.1	4.3	550	9.7

7 DISCUSSION OF THE RESULTS

7.1 TPS Rounds to Find the Mean

Interestingly, after turning 100 sets to find the mean, this only proved that turning 4 sets in a round gets within ±2" of the mean. From studying the data collected, it would appear that 20 rounds are required to get a good approximation of the mean. Turning 6 or 10 has little if any benefit over 4, but it does afford the extra data to allow bad observations to be culled. This is most likely due to error theory, knowing that using 4 observations of equal quality doubles the accuracy but to see that improvement again requires 16 observations. This can be seen by looking at Figure 5.

7.2 TPS Instrument Errors

Oddly enough using the same data Table 3 does not agree with this. From the coordinates there would seem to be no advantage turning 4 sets over a single FL/FR. Yet there is a distinct advantage in turning 16 sets (20 used here; however the math would suggest that 16 are enough). Looking at the data, patterns can be seen in the reference object data (being station 1002) that the instrument seems to read in blocks. This seems to be related to the way the distance is determined more than the angle.

7.3 TPS Use in the Field

Moving to a more practical point of view, using the coordinates collected over several sessions and comparing them, the TPS seems to yield approximately the same errors as it does in a setup free environment. This would tend to indicate that the instrument error and setup errors are of similar magnitudes and sometimes work together and sometimes against each other. In the end it all averages out. From the data collected, by comparing Tables 3-5, it would seem reasonable to expect an error of ±5 mm on lines less than 500 m in length. If traversing using constrained centring on shorter lines, there is a small decrease in error, i.e. 3 mm over 150 m or 4 mm over 250 m. For the ease of field use and the random chance observation, it is good to allow 5 mm per station. This will also allow the surveyor to re-occupy this station at another time using a different setup with the knowledge that the error circle for that station will not change.

7.4 GNSS Use in the Field

Now that a benchmark for comparison has been determined (being the TPS), it is possible to look at the initial question: How does GNSS measure up when determining angles? From the collected data, the results in Table 7 would indicate that with enough measurements GNSS is able to determine good angle measurements. This is true of any instrument, take enough readings and you will be bound to get a good answer. However, that is not what we set out to determine. Table 7 is the result of the mean of many observations. With that in mind, GNSS can approximate the TPS results but the data collection and processing is a monumental task.

7.5 GNSS vs. TPS Used in the Field for Angles

Using normal practice and comparing the results in Table 8 indicates that GNSS is not able to yield similar results to a TPS over short distances in a direct head-to-head comparison, bearing in mind that these results are the mean of several days of observations with the GNSS and TPS measurements using the same setup on the same day. As will be discussed later, this

does not preclude GNSS being used over short distances.

7.6 GNSS Coordinate Repeatability

From the data collected, the results in Table 6 would indicate that static GNSS is about twice as accurate as Real Time Kinematic (RTK) GNSS, which is well studied and estimated to be ±30 mm at 95% (Gibbings and Zahl, 2014). This indication of accuracy is in line with industry held beliefs (unpublished). Therefore, with four times the readings, RTK will approximate static GNSS, which when compared to Table 4 is about three times as large as TPS errors (95%) over lines under 500 m.

7.7 Practical Assessment of GNSS for Cadastral Surveys

From all of the discussion and results above, one thing is very clear: static GNSS is no match for TPS over short lines in a head-to-head comparison. To clarify, using a ±15 mm error as the test scenario, a TPS achieves this error in 775 m as a single FL/FR although that would degrade the traverse if the standard procedure is to use rounds of 4 sets. Using 4 sets, a TPS will achieve foresights of 1,550 m before reaching the static GNSS error of 15 mm. However, this does not preclude GNSS from being used over short distances. Armed with the knowledge that a TPS used with 4 sets in a round produces a ±5 mm error for every line (foresight) measured, it does not take long before the cumulative error reaches ±15 mm. This could either be one very long (in cadastral terms) line and a traverse station or three foresights of any distance. This could mean it is quite possible to measure a very short line with GNSS if it is not possible to measure it with traditional methods in less than three foresights. The problem then faced is how a backsight is selected for any work to be performed with a TPS using the GNSS-observed station. For this the only reasonable proposition is to take 'a normal-length traverse leg length' for that type of location, multiply it by 3 and place another GNSS point that far away. It will be most likely that an actual traverse will be needed to join the points together. However, sometimes in clear conditions, a single leg of three traverse lengths will work just as well. The concept here is not a direct head-to-head comparison but what distance would be covered using three foresights with traditional methods. In the end, we need to consider the size of the error ellipses at the points we are trying to determine.

8 HOW TO APPLY THE REGULATION

Using GNSS to traverse over or around a problem is not difficult to include in the Surveying and Spatial Information Regulation as it just forms part of the traverse calculation like any other line. The important thing here is that the close needs to be performed before the least squares adjustment. Adjusting poor data only leads to poor results. Some would argue that the adjustment will identify poor results, but how does that get related to the Regulation? This also requires a traversing methodology to be applied to GNSS work rather than a network methodology, so each line is measured and the closing angle needs to also be measured to determine the angular misclose. GNSS is predominately used by geodetic surveyors for different applications and hence these have driven its development. There needs to be an understanding of how GNSS can be used for cadastral work and compared to the Surveying and Spatial Information Regulation 2012, allowing the best methodology for GNSS to be used.

9 CONCLUDING REMARKS

The intention of this paper was to determine what line length would equate GNSS to TPS for use with regards to cadastral surveys. The theory behind this was to determine the shortest line between two points measured by GNSS that would compare to TPS measurements to maintain the azimuth of a survey without introducing a swing due to lesser accuracy measurements. This has been achieved and as expected, in a head-to-head comparison the distances are not practical for cadastral survey work. After further consideration of the data collected and considering the process of cadastral survey, it became obvious that determining the head-to-head comparison was not the only way to determine the end goal of not introducing an azimuth swing. This was to understand the cumulative effects of traversing.

While this paper cannot be used as statistically correct representation of the errors due to the lack of field data, there is sufficient data to draw the conclusions that have been drawn and there is significantly more data than would be collected on any one job that would use the process of combining both GNSS and TPS techniques. What this paper does is identify the capability of GNSS to be used within cadastral surveying in the appropriate situations. The other thing that this paper highlights is the need for further statistically sound research to be carried out in order to define error circles in traversing without using adjustment to determine them. This research should include both GNSS and traditional traversing.

The main conclusions of this paper are:

- Static GNSS has a point error of approximately ±15 mm at 95% and is about twice as accurate as RTK GNSS, which has been determined by previous research to be ±30 mm at 95%.
- A current 3”, 3mm + 3 ppm total station has a point error of approximately ±5 mm at 95%, being 3 times better than static GNSS over distances of less than 500 m.
- Therefore, it is reasonable to use static GNSS points at spacings of 3 foresight lengths (which is dependent on the location) without adding any azimuth swing into the survey.

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Contract Resources for RMS Survey Investigations

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ABSTRACT

The increase in outsourcing technical work provides challenges in terms specifying technical requirements that are fit for purpose. Those commissioning the work need to use an efficient method of scoping the work, and provide sufficient detail to ensure the desired product is delivered. Those providing the survey services need to fully understand the required deliverables (including traceable quality assurance processes). RMS Surveying has updated specification G73 Detail Survey in CADD Format and the associated guide, RMS CADD Manual for Surveying. In a risk management context, insufficient or incorrect information to support the design process leads to very costly delays and variations that can flow into the construction phase. It is agreed that fundamental survey methods are common to both public sector and private surveyors. However, for infrastructure development, an understanding of the inherent 3-dimensional tolerances of a given feature is required. This presentation will outline these requirements along with the structure of specification G73 and the CADD manual to enable efficient contracting and as a training resource for the industry. APAS2015 will provide a vehicle for stakeholder engagement and feedback in this regard.

KEYWORDS: *Specification, topographical surveys, contracting*.

Plantation Management and the Forest Life Cycle: Spatial Solutions to Non-Spatial Problems

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ABSTRACT

Forestry is defined as the practical application of scientific, economic and social principles used in the establishment and management of forests. The Forestry Corporation of NSW is the largest manager of commercial native and plantation forests in NSW with more than two million hectares under management. These plantation forests have been planted with specific varieties of trees for the purpose of timber production and make up approximately 250,000 ha within the state forest estate. Softwood plantations comprise about 85% of this area with radiata pine being the dominant species in the cooler parts of the state, and minor species and hybrids grown in the northern regions. All of these varieties follow the forest life cycle from establishment to harvesting and re-establishment within each rotation. The Corporation is committed to ensuring the supply of timber from NSW state forests today and into the future, while also protecting other forest values such as biodiversity, clean air and water, and public access for recreation. These competing interests present several challenges for forest planners, harvesting supervisors, haulage schedulers and fire protection managers. Spatial information is used on a daily basis within the forest management process and assists with providing answers to key forest management questions. Several of these solutions include the use of Global Navigation Satellite System (GNSS) tracking to monitor monthly harvesting operations and the use of slope class analysis to determine the safe working limits for harvesting machine types. This paper will outline several of the spatially related solutions being used by the Corporation within the forest life cycle as applicable to softwood plantations in NSW.

KEYWORDS: *Forestry, plantations, GIS.*

1 INTRODUCTION

The Forestry Corporation of NSW (the corporation) manages more than two million hectares of native and plantation forests for the economic, environmental and social benefit of the people of New South Wales. Its purpose is to (1) ensure the immediate and ongoing production of timber today and into the future, (2) protect other forest values such as biodiversity, clean air and water, and (3) provide public access for recreation (Forestry Corporation, 2015).

Competing interests are dealt with on a daily basis through the corporation's role as not only a timber producer but also a land manager. This paper provides an overview of the corporation's softwood timber production activities, summarises the forest life cycle as applicable to those activities, and outlines several of the methods used which enable the corporation to meet its purpose.

2 SOFTWOOD TIMBER PRODUCTION

Plantation forests are those areas that have been planted with specific varieties of trees for the purpose of timber production. Radiata pine forms the basis of the corporation's plantations on the tablelands and in the southern areas of NSW with hybrids that are more suited to a warmer climate being used in the northern forests around Grafton (Figure 1). The softwood plantation estate totals just over 200,000 ha with approximately 3% being harvested and re-established annually, producing 3.2 million tonnes of logs.

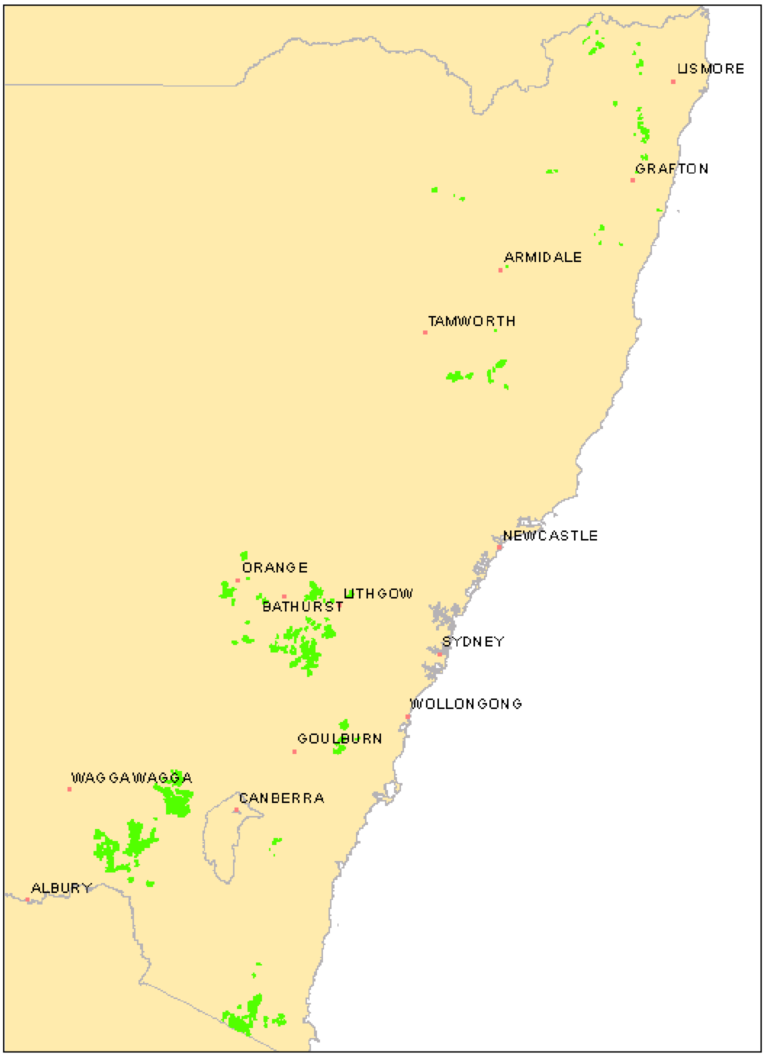


Figure 1: Forestry Corporation softwood plantations.

3 FOREST LIFE CYCLE

Each planted rotation follows a generally predetermined process known as the forest life cycle (Figure 2) and takes approximately 30 years to complete. This section provides an overview of the relevant processes in the forest life cycle.

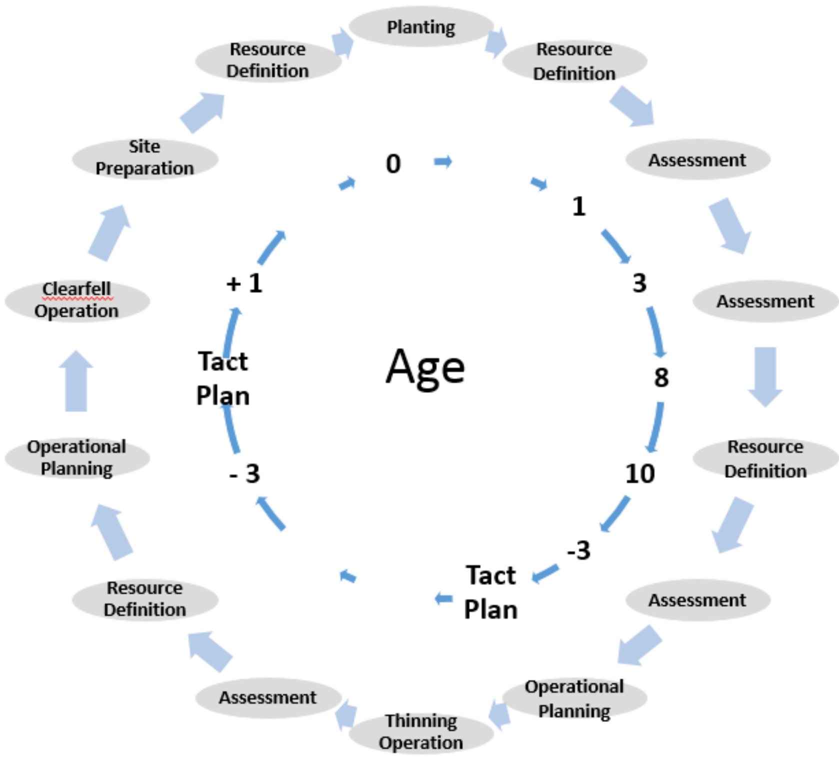


Figure 2: Forest life cycle.

3.1 Age 0: Planting

The young seedlings arrive at the forest ready for planting after being grown at the nursery for approximately 8 months. Seedlings are organised into trays according to the nursery location, seed type and overall genetics. These details are recorded against the areas established and form the basis of the historical records for the life of the crop. Additional records will be added over time with each significant event being recorded within the forest management system.

The seedlings are planted in rows (Figure 3) according to the establishment plan – this takes place in the coolest quarter of the year, when the seedlings are dormant and the ground is moist. Fertiliser is applied, giving the seedlings a growth boost in order to counter competing vegetation. In the 2014 planting season, the corporation planted almost 9.3 million seedlings across NSW.

Figure 3: Planting operations.

3.2 Resource Definition

Defining the spatial extent of the crop initially occurs at the conclusion of planting. The redefinition process occurs several times throughout the life of the crop:

1. At age 8, prior to age 10 assessments and inclusion in the tactical plan.
2. At the conclusion of thinning operations.
3. At age 20, prior to age 23 assessments and inclusion in the tactical plan for clearfell.
4. During the operational planning stage.
5. At the conclusion of site preparation activities, which define the available planting area.

A combination of GPS and remotely sensed data (Figures 4 & 5) is used during this process with the desired accuracy for the outside perimeter being ±4 m. Light Detection and Ranging (LiDAR) data has also been found to be particularly useful in the re-mapping of roads and drainage lines.

Figure 4: Resource boundaries prior to redefinition.

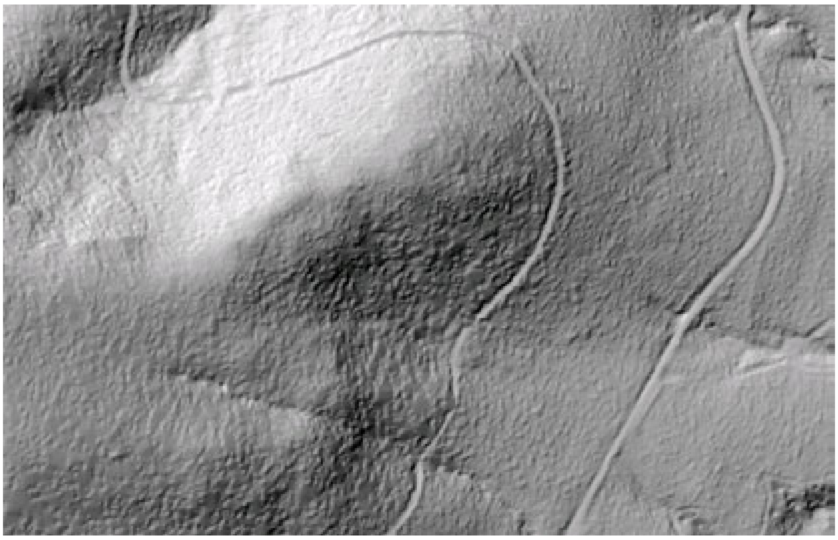


Figure 5: LiDAR-generated Digital Terrain Model (DTM).

Trials have begun in the Canobolas State Forest using LiDAR data to generate areas of ‘difficult movement’, i.e. thick vegetation between 0.5 and 1.5 m in height (Figure 6). These areas are being assessed for weed infestation (e.g. blackberry and wattle) and will be classified as not suitable for re-establishment until the weeds are appropriately treated.

Figure 6: LiDAR-generated difficult movement areas.

The corporation has been developing an iDevice application for use by staff for mobile data collection. Modules specifically designed for use during fire fighting and for the collection of biological data by ecologists allow for nightly downloads whenever in mobile phone range. This data is integrated with the corporate and regional datasets once verified.

3.3 Assessments

Assessments are undertaken at various stages within the life of the crop (Table 1) with the initial assessment used to measure seedling and planting quality, and to estimate seedling survival. Future assessments are used to assess the crop for product allocation whilst also estimating stocking per hectare and timber volume at maturity. The assessment locations are randomly placed within the stand and provide a valuable indication of the state of the forest.

Table 1: Assessment schedule.

Survival
Age 3
Age 10
Post Harvest – Thinning
Age 23
Pre Harvest – Clearfell
Residue (post Clearfell)

3.4 Operational Planning

Prior to harvesting operations taking place, the planning team for the region undertakes an operational planning process in order to ensure that the operation is completed sustainably and economically. The factors considered during this phase include:

1. Method of harvesting and number of machines involved/required.
2. Access requirements for harvesting machines.
3. Neighbours.
4. Movement of product to roadside.
5. Roadside storage of timber, i.e. size, design and composition.
6. Movement of product to customer, i.e. state of roads – gravelling, additional drainage, B-double access and permissions, swept path analysis for intersections.

Additional factors will influence the seasonal timing of the operation. These include slope, geology and soil type (moisture retention), and understorey vegetation (weeds) which influence trafficability and time of the year that harvesting machines may be able to safely work on the site. All spatial information related to the operational site and forest location is reviewed during this stage.

3.5 Thinning Operation

When the trees are about 14 years old, the stand may be thinned in order to allow the stronger trees more room to grow (Figure 7). The remaining trees get more light, nutrients and water with the harvested trees being used for pulp, which is used in newspapers or as chip for particle board.

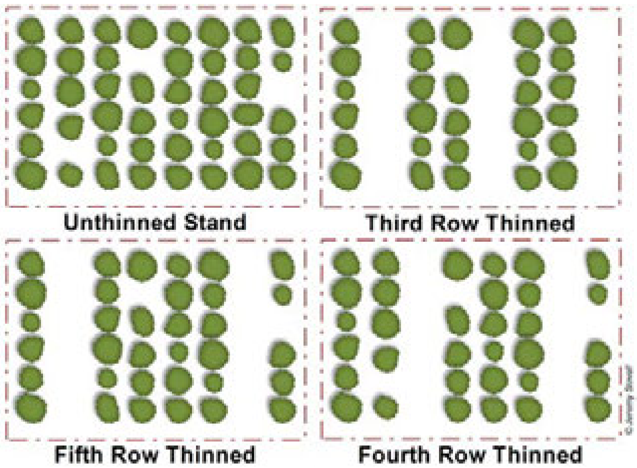


Figure 7: Harvesting options for thinning operations.

At the conclusion of the operation, the affected resource is remapped to define the boundary between the thinned and unthinned stands. This is required to accurately determine the products available at clearfell within the remaining trees.

3.6 Clearfell Operation

The tactical planning process (see section 4) identifies the optimum time for harvesting that sustainably meets the product delivery commitments of the corporation. Harvesting the site during a clearfell operation involves more than just cutting down the trees. Each log is cut up according to a schedule depending on the customer's requirements, e.g. 480 t @ 6.1 m long, 120 t @ 4.2 m long. Where fitted, the harvesting operation can be followed using the GPS track, which records the cuts made by the harvester (Figure 8).

Figure 8: GPS-recorded harvesting cuts with remaining patch.

The logs are moved to the roadside and placed in stacks (Figure 9) according to delivery point and product configuration. At the conclusion of the clearfell operation, the remaining resource is remapped to assist in determining the available plantable area that will undertake site preparation activities.

Figure 9: Logs at the roadside.

3.7 Site Preparation

At the conclusion of the clearfell operation, the area is assessed for its re-establishment suitability with a program of works being undertaken depending on topography, proximity to major drainage lines and sensitive vegetation, soil type, previous rotations, and overall forest health. These factors will influence the design of the planting programs and planned stocking rates. An area with a planned stocking of 900 seedlings per hectare will be prepared differently, possibly due do its steep topography, compared to a flatter area with a planned stocking of 1,100 seedlings per hectare. During this stage the corporation’s nurseries are also growing the seedlings, which are sown in early September in preparation for next year’s planting season.

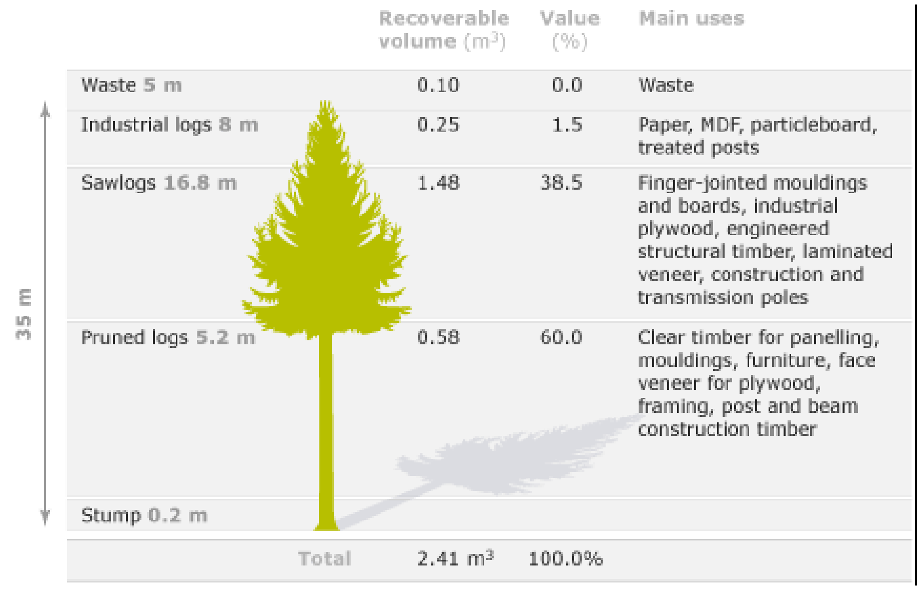
Herbicide is applied to reduce competition with additional planning being undertaken to take future access and fire protection requirements into account. GPS tracks are recorded for all operations in site preparation to assist in the determination of the plantable area and the total number of seedlings required from the nursery.

4 TACTICAL PLANNING

Each of the corporation's customers requires different products depending on their section of the forest product industry. These products (Figure 10) come from different sections of the tree, which can be categorised into three main parts:

1. Log suitable for sawn products (Figure 11).
2. Log suitable for pulp.
3. Biomass waste.

A tactical plan is created using the Woodstock modelling software, which predicts the sustainable cutting strategy for the next 70 years.



	Recoverable volume (m^3)	Value (%)	Main uses
Waste 5 m	0.10	0.0	Waste
Industrial logs 8 m	0.25	1.5	Paper, MDF, particleboard, treated posts
Sawlogs 16.8 m	1.48	38.5	Finger-jointed mouldings and boards, industrial plywood, engineered structural timber, laminated veneer, construction and transmission poles
Pruned logs 5.2 m	0.58	60.0	Clear timber for panelling, mouldings, furniture, face veneer for plywood, framing, post and beam construction timber
Stump 0.2 m			
Total	2.41 m^3	100.0%	

Figure 10: Potential products at clearfell.

Figure 11: Potential sawlog products.

The tactical plan (Figure 12) relies on accurate yield predictions, which in turn rely on accurate areas, accurate growth models and yield tables, and accurate stocking.

Additional factors influencing the plan include:

1. Redefinition activities, e.g. remapping and assessments.
2. Proximity to other harvesting operations.
3. Delays due to wet weather.

4. Resource becoming unavailable, e.g. fire.
5. Customer's required product ratios.
6. Timing of re-establishment.

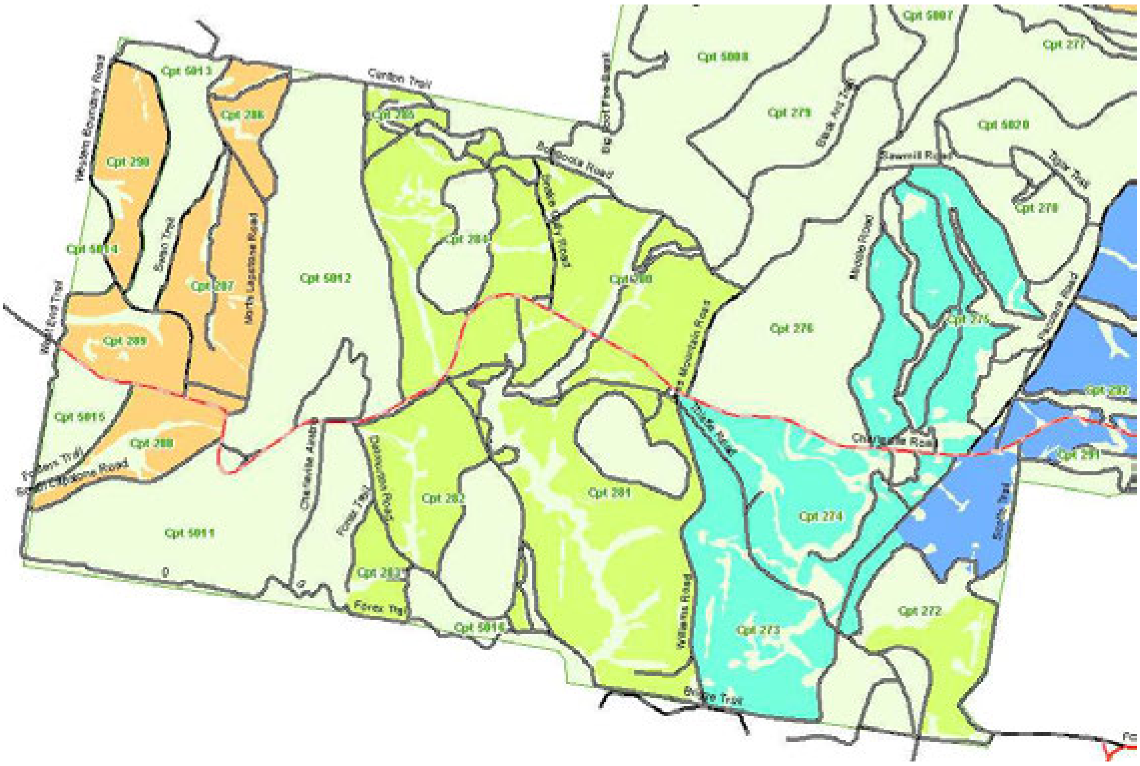


Figure 12: Typical tactical plan breakdown by year.

5 LAND MANAGEMENT

As a land manager, the corporation is mindful of its ongoing responsibilities and regularly liaises with stakeholders to address land management issues as they arise. A selection of the current issues is listed in Table 2. The regional stewardship team also monitors the current and future operational needs of the organisation and produces a series of entry restrictions and exclusion maps for the safety of the general public.

Table 2: Land management issues.

Pest Management • Weed spraying • Baiting programs	**Monitoring** • Illegal dumping • Water quality	**Neighbours** • Noise • Road access • Fencing • Encroachments
Public Access • 4WD • Motorbike • Bushwalking • Bird watching • Mushroom picking	**Organised Events** • Orienteering • Car rally	**Permits** • Firewood collection • Fossicking • Grazing • Bees • Mining
Fires • Hazard reduction	**Fauna** • Endangered	**Flora** • EECs

6 SELECTED SPATIAL SOLUTIONS

6.1 Wood Movement

One of the questions regularly asked of the Geographical Information System (GIS) team relates to predicting the movement of wood around the state depending on the changing needs of our customers. For example, how many tonnes of wood are moved over a particular section of road over the 6 years of the tactical plan? Whilst the answer is not a spatial one, it requires spatial solutions to enable its answer.

This particular problem was tackled using a multi-step process. In summary:

1. The tactical plan defined the areas to be harvested using both thinning and clearfell operations. Yield information was assigned to loading bays by customer and harvest year.
2. Roads were assessed according to their suitability for haulage:
    a. Surface
    b. Lanes
    c. Grade / visibility limitations
    d. B-double permissions – RMS, NHVR
        i. General access
        ii. 19 m B-doubles operating at full axle loads
        iii. 23 m B-doubles
        iv. 25/26 m B-doubles
3. The roads were assigned to a transport network in ArcMap.
4. The customer destinations were allocated to also calculate lead distances for haulage costs.
5. Models were created in ArcMap, generating the routes using the Network Analyst extension which also assigned the yields to the route.
6. A new feature class (Figure 13) was created using a model that summed the yields over each line segment of the network.

Figure 13: Tactical plan with predictive volume analysis for period F15-F20.

6.2 Site Safety

Softwood forests are usually sighted in areas that are often described as mostly undulating and sometimes steep. The safe working limits of harvesting machines are a determining factor in deciding on appropriate machine type and use within a harvesting site. LiDAR-generated slope information is used to determine the percentage of area within certain slope classes for each product type. These slope classes are also used to determine contractor payments as different rates are applicable depending on the machines used and the slope of the plantation area.

7 CONCLUDING REMARKS

The Forestry Corporation of NSW is responsible for the sustainable production of timber within its plantation forests as well as protecting forest values and providing access for the public. This paper has summarised the forest life cycle in relation to the corporation's operations whilst also outlining the current land management issues and spatial solutions being used to provide solutions on a regular basis.

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Mean High Water Mark: Is the Mean the Answer?

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ABSTRACT

Raising issues in respect to the determination, appropriateness and even validity of Mean High Water (MHW) and surveyor's methodologies in determining the Mean High Water Mark (MHWM) boundary has been a part of APAS since its inception; first with "Mean High Water Mark Revisited" in 1995 and then "Accurate Mean High Water determination – Fact or Fiction" in 2005. This current paper could be considered as chapter 3 in the trilogy. In 1995, the awareness of how MHW came about in relation to property boundaries was raised and whether both surveying practice and the definition in use complied with the intent prescribed by the legal profession in describing the land/sea boundary. Then in 2005, the accuracy of survey practice methods for determining MHW was brought into question. Now in 2015, the validity of determining the mean itself in MHW is to be examined. Tidal observation data is utilised to bring forward the 1995 issues testing the mean determined by the data used by surveyors against the legislated definition and early legal profession expectations to see if they all are in agreement or if there are disparities. The paper then looks toward the future by throwing sea level rise into the mix. Are we (surveyors) doing it right and will current practice be appropriate in the future? Or does something need to change? The findings presented indicate that there are disparities between practice, legislation and expectations. Introducing sea level rise creates further issues. For surveyors to answer the needs of the future in respect to determining tidal boundaries, it would seem that the how and the definition of MHWM may need to change.

KEYWORDS: *Mean High Water Mark, definition uncertainty, sea level rise, survey practice.*

1 INTRODUCTION

The riparian title boundary Mean High Water Mark (MHWM) should be familiar to most surveyors who work on the coast of NSW. So should the definition of Mean High Water (MHW) be familiar. But does any surveyor really understand what they are undertaking when defining the mean high water mark? Do they fully comprehend the implications of the definition and whether their work practice conforms to the definition?

But is it just the surveyor? What about the learned judges who have handed us down the definition of mean high water? Did they fully understand what they were doing when they tried to put into words a definition of the particular position along the coast that they were trying to define? Does the definition prescribe the boundary they considered to be the limit of the land? Now throw in sea level change, rising sea levels if you are a believer, and consider whether current practice reflects such an event. Does the definition provided by the legal profession also reflect such an event? Does current land title in NSW consider and/or reflect sea level change?

At the 1st APAS conference, Blume (1995) asked similar questions about the definition of MHW and cast similar doubts as to the validity of current practice to conform to the definition. A decade later, at the 11th APAS conference, Songberg (2005) closely examined current practice. What that investigation uncovered was that the various recognised methods did not provide repeated accuracy in defining MHW. A variability of at least ±0.1 m in level was likely between determinations.

Another decade has passed, and now at the 20th APAS conference in 2015 surveyors are still conducting MHWM definitions the same way. But now the issue of sea level rise is starting to be considered in other areas, particularly in how it is likely to impact on title. Perhaps it is time to again look more closely at the definition of MHW and to see if some of the questions, initially raised 20 years ago, can be answered and to see if survey practice needs to be refined to include sea level rise.

2 MEAN HIGH WATER DEFINITIONS

The current definition of mean high water (mark) that surveyors should be acquainted with is stated in the Surveying and Spatial Information Regulation 2012 (NSW Legislation, 2014): "*the line of mean high tide between the ordinary high-water spring and ordinary high-water neap tides*". This definition has its roots in *Attorney General v. Chambers* (1854). The case, from the south coast of Wales, involved the Attorney General taking action to retain the seashore for the Crown (then Queen Victoria) and prevent the land being used for coal mining. The judges came to the conclusion that the limit of the land was "*the medium high tide between the springs and neaps*". Although ordinary (as in ordinarily occurring tides) was a consideration in this deliberation, it was not part of the definition handed down. The term ordinary was introduced later (Blume, 1995) in the case of *Tracey Elliot v. Morley (Earl)* (1907), which referenced the mean as the average of the medium or ordinary tides (Corkill, 2013).

Another definition in current use is found in the Manual of the New South Wales Integrated Survey Grid (ISG) (NSW Department of Lands, 1976): "*the mean of all high tides (including both spring and neap tides) taken over a long period*". This definition is distinctly different from the surveying regulation definition as it specifically includes all tides and potentially could provide a different result depending how literal the interpretation is taken into survey practice.

These two definitions are not the only ones as many other variants exist and/or have existed over time, including within the surveying regulations (Blume, 1995). Does either, or any, answer to the intent of the boundary between the Crown and private lands handed down in the judgment of *Attorney General v. Chambers*? Does any exemplify current survey practice? Is it what society expects? Whichever definition is considered the most relevant, the surveyor must first come to grips with the spring and neap tides and how observation practice must relate. So, what are spring and neap tides?

3 MEAN HIGH WATER DEFINITION INTERPRETATION

In order to satisfy a criteria imposed by definitions, the surveyor must first come to terms with the meaning of the words. The definition of mean high water is no different and the result

could vary depending on the interpretation perceived. It is clear that the two definitions of MHW given at the start are very different and could potentially give different results. Even within a single definition, interpretations could vary producing varied results. At first what seems quite clear may not necessarily be so.

3.1 Spring and Neap Tides

Spring tides have nothing to do with the season spring, as they occur all year round. Simplistically, spring tides occur around the new and full moons when the earth, moon and sun align and neap tides occur when the moon is perpendicular to the earth-sun alignment (Figure 1). There are many other factors, including oscillations in the earth-sun-moon alignment, which create variations in the tides and the cycles between the tides, a discussion of which is beyond the scope of this paper. The simplistic view, however, will suffice.

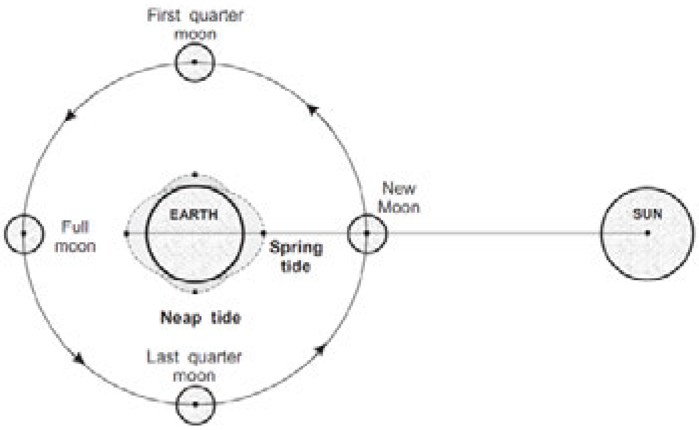

Figure 1: Spring and neap tides by moon phase (Couriel et al., 2012).

Both Figures 1 and 2 suggest that a spring or neap tide is a singular event with the spring tide being larger than the neap tide. Both diagrams suggest that the spring and neap tides occur at the time of the particular moon phase. An important point to note is the moon cycle, which is 29.53 days long, making each quarter 7.38 days (on average as in reality the duration varies slightly). The duration between spring and neap tides, in accordance with the moon, should thus be just over 7 days. Does the mean of high tides from one moon phase to the next provide a meaningful value for mean high water?

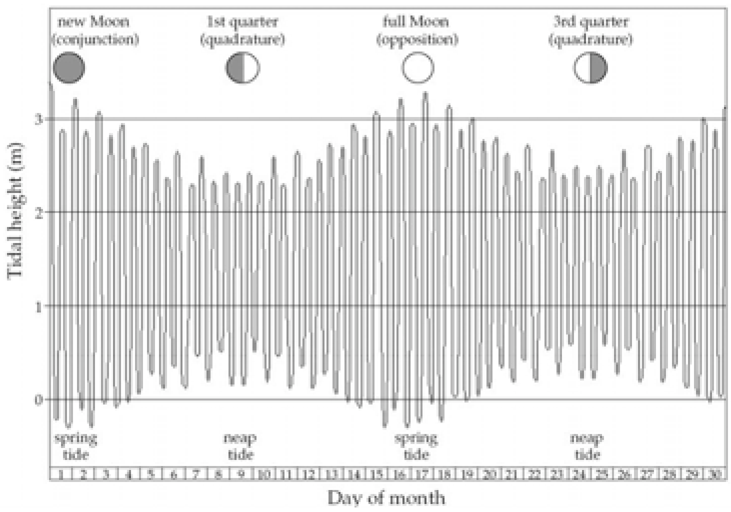

Figure 2: Spring and neap tides range (Earth Science NHS, 2015).

Dividing 12 months of tidal data into segments according to the moon phase and taking the mean of high tides from one moon phase to the next results in MHW values according to Figure 3. The result is highly varied with a 0.3 m range from the highest result to the lowest. This would mean that MHW could vary by at least 0.3 m from one week to the next and the lateral MHWM boundary could vary tens of metres over flat terrain. Such a result is not likely to have been envisaged by the proponents of MHW as a measure for a land boundary. Certainly the society of today would not consider such a variable entity as being desirable, they would not know from one week to the next where their boundary was located.

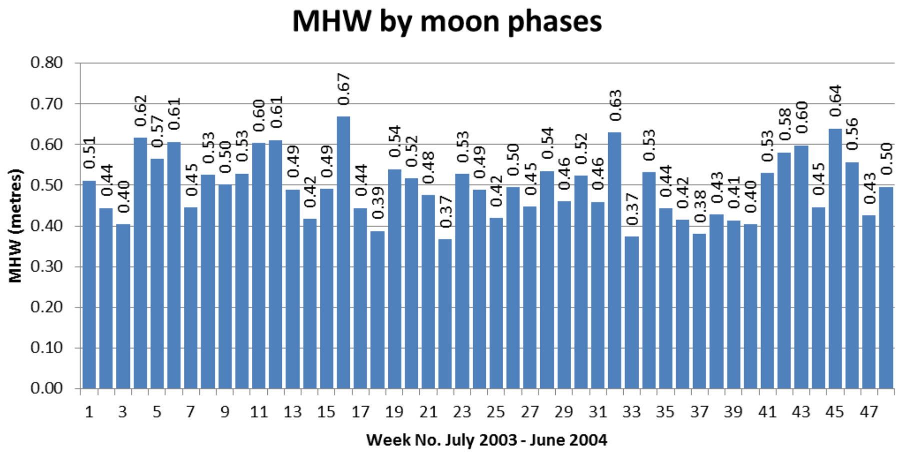

Figure 3: MHW at Port Macquarie in July 2003 – June 2004 by quarter moon cycle.

If the timing of the moon does not produce a desirable outcome for determining when the spring and neap tides occur then what other options are there for determining such necessary events as required by the surveying regulation definition? Other information suggests that spring and neap tides are not singular events but instead multiple events ranging over a period of tide behaviour. Some texts (e.g. Marine Science Australia, 2014; Port of London Authority, 2015) indicate spring high tides are typified by being higher than average and neap high tides are lower than average (Figure 4).

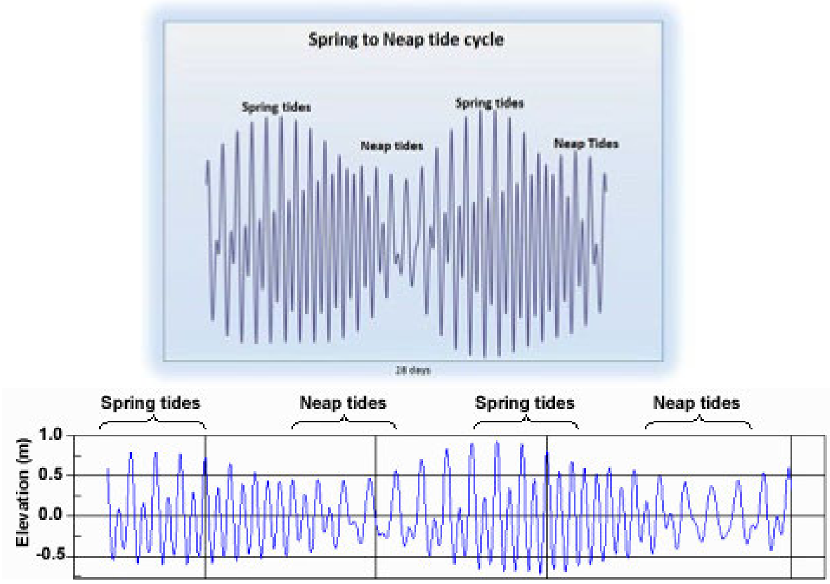

Figure 4: Spring and neap tide duration (Marine Science Australia, 2014; University Corporation for Atmospheric Research, 2007).

Examination of Figures 3 and 4 shows that even within a spring tide cycle the range between successive tides can be smaller than average so that classifying a spring or neap tide by an individual tide event can be problematic. It would appear that to determine spring or neap tides, one must stand back a little and examine the overall picture.

The exact timing of the spring and neap tides in relation to the moon phase in such a multiple event is also not exactly determinable. It can only be determined that the spring tides occur around the time of the full and new moons and the neap tides occur around the time of the 1st and 3rd moon quarters (Figure 5). Although only full and new moons are shown (for clarity), Figure 5 also suggests that the timing of the peak spring tides in relation to the moon phase can be before, during or after the moon event.

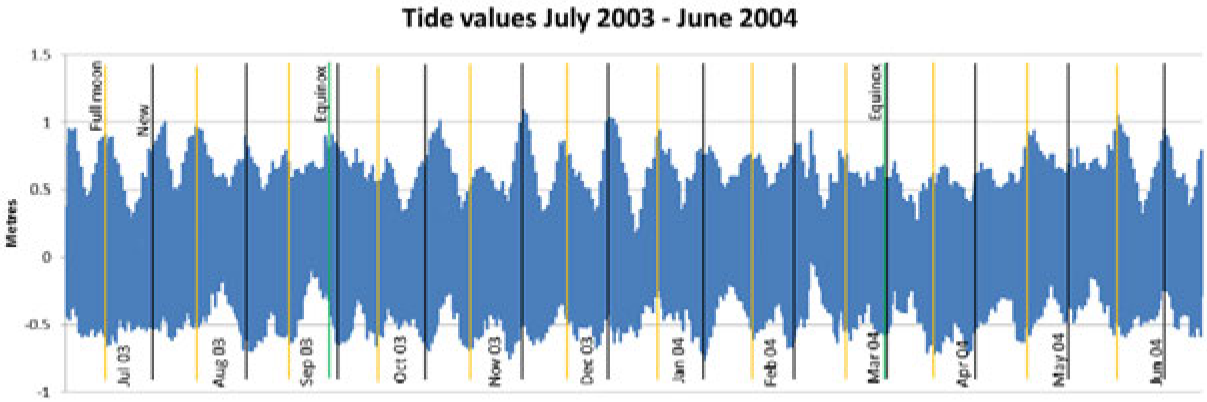

Figure 5: Tide values to moon phase at Port Macquarie in July 2003 – June 2004.

However, Figure 6 specifically indicates that the spring and neap tides occur shortly after the moon phase and only in a short interval. Figure 6 also supports the multiple-event scenario for spring and neap tides but like previous diagrams does not give a figure. The only indication is from the text which also indicates that spring tides are when the highest and lowest tides of the month occur and the neap being when the lowest high tide and highest low tides occur (VisitMyHarbour.com, 2015). This is in contrast to Figure 1, which indicates that they occur at the time of the moon phase.

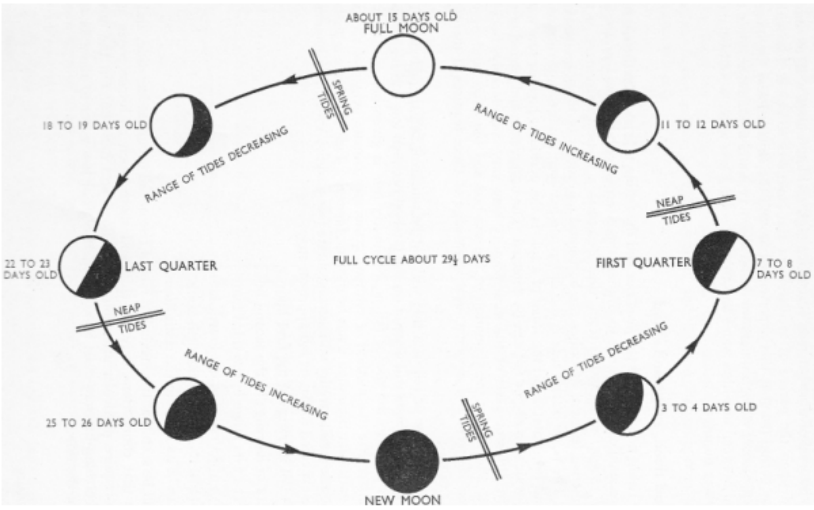

Figure 6: Spring and neap tide timing to moon phase (VisitMyHarbour.com, 2015).

Casting the information net even wider, dictionaries provide even more insight into what are spring and neap tides.
Spring tides:

- dictionary.com – either of the two tides occurring just after the full and new moons where the difference between the high and low is the greatest.
- thefreedictionary.com – a tide in which the difference between the high and low is the greatest occurring just after the full and new moons.
- oxforddictionary.com – a tide just after a new or full moon where there is the greatest difference between the high and low water.
- macmilliandictionary.com – a tide when there is a big difference between the highest and lowest levels of the sea.

Neap tides:

- dictionary.com – tides having less than half the range of spring tides occurring around the first and last quarters of the lunar cycle.
- thefreedictionary.com – tides midway between spring tides that attain the least height.
- oxforddictionary.com – a tide just after the first and third quarters of the moon where there is the least difference between the high and low water.
- macmilliandictionary.com – a tide that has the least amount of change between the highest and lowest levels of the sea.

As it can be seen, even the various dictionaries are not in complete harmony as to what is a spring and neap tide. Just how close to reality the definitions are is yet to be seen. From the dictionaries the most common thread is that spring tides have the greatest range between consecutive high and low tides whereas the neap tides have the least range. Utilising the data that constituted Figure 5, the high and low tides can be identified, the differences calculated and the relation to the moon phases determined. The results of such a comparison can be found in Figure 7, which shows two months of the 12-month period of data.

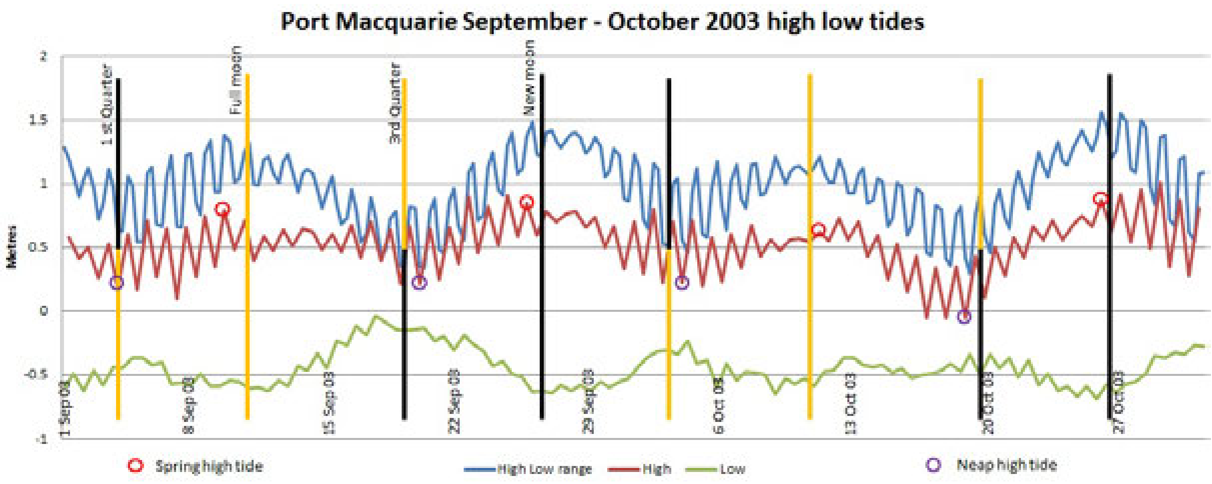

Figure 7: High low tide and range compared to moon phases at Port Macquarie in September-October 2003.

It does not take long to see that none of the dictionary definitions completely satisfies what is found in reality. Assuming a single definition that the spring tide has the greatest range between high and low and the neap the least range and correlating the appropriate range with the high tide, it can be seen that the high tide event that corresponds to a maximum spring range does not always occur after the moon phase. Sometimes the high tide occurs before the moon. The situation is similar with the neap tide event. Also the high tide associated with the greatest high low range is not necessarily the highest tide in the period. The neap tide also

exhibits a similar trait. The neap tide event does not necessarily occur with lowest high tide.

There is an immense amount of literature that gives explanations as to what and when are spring and neap tides. The above is just a sample and it shows that there is no one distinct answer. It would seem that spring and neap tides are nebulous titles given to tides that are, for spring, high tides that are larger than most coupled with low tides lower than most and have a larger than average tidal range occurring around the full and new moons, and for neap, high tides smaller than most coupled with low tides higher than most and have a lower than average tidal range occurring around the 1st and 3rd quarter moons. Because the definition of the mean in MHW is synonymous with spring and neap tides, a surveyor is faced with difficulties in identifying, in any positive manner, particular spring and/or neap tides as part of the mean determination process.

3.2 Ordinary Tides

The Surveying and Spatial Information Regulation adds another dimension to determining mean high water. It specifically requires only ordinary spring and ordinary neap tides to be taken into consideration in determining MHW. But what are ordinary tides? Lord Chancellor in *Attorney General v. Chambers* (1854) possibly provides the best insight into ordinary tides: "*The right* (the Crowns right to ownership) *is confined to what is covered by ordinary tides, whatever the right interpretation of that word. What is the meaning of the word ordinary? It is evidently a word of doubtful import*."

The judges in their deliberation did raise the issue of whether some tides were considered ordinary or not: "*There are the spring tides at the equinox, the highest of all. These clearly are excluded … for though in one sense they are ordinary, i.e. according to the usual order of nature … they do not ordinarily happen, but only at two times of the year*."

From the deliberations it may be possible to consider that the spring tides, especially the higher the king tides, are not ordinary in that the position on the foreshore covered by a spring tide is not ordinarily reached but only does so during short intervals. But what then are ordinary neap tides. Various literatures always seem to gravitate back to *Attorney General v. Chambers* and the heavy reliance on the 17th century treatise "De Jure Maris" by Lord Chief Justice Hale. The neap tides were considered by Justice Hale as being ordinary tides. This would presume than that all neap tides were ordinary tides. The judges in *Attorney General v. Chambers* however reasoned that for the same reason the highest spring tides would be excluded so too should the lowest high tides for they happen as often as each other. It would appear that the calculation of MHW does not include the higher monthly spring high tides or the lower monthly neap high tides (Corkill, 2013). The problem though is how to identify the particular spring and neap tides that could be considered to be not ordinary.

The allocation of the highest spring tides to the equinoxes in March and September may be true somewhere but not in the observations shown in Figures 5 and 8. Here, there are eight high (spring) tides of 1 m or greater mostly in November and December which is as expected in Australia and not around the time of the equinoxes. But two of the higher high tide events occurred in August and June. Most occur about the time of the new moon; however the higher tide on 3 June occurs around the full moon. A choice of 1 m as an indicator of a higher or non-ordinary high tide is completely random and does not create a justification criterion for exclusion. There are a few other high tide events falling just short of 1 m in July, August, January and May around the time of the full moons that could equally be considered as non-

ordinary events. The other issue is that MHW is an ambulatory entity. It moves up and down over time. Limiting the measurement of the high tide event places an artificial restraint and barrier to the natural movement of the tide. Such a restraint may even contravene the doctrine of accretion and erosion.

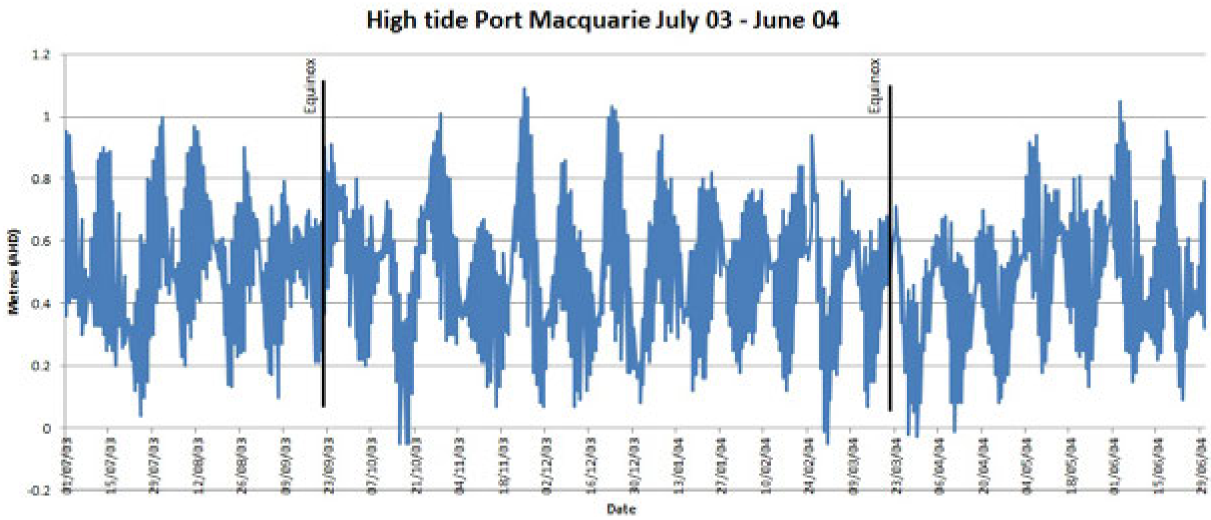

Figure 8: High tide values at Port Macquarie in July 2003 – June 2004.

It may not necessarily be the higher tides that could be considered as not being ordinary. If Figure 4, 7 or 8 is examined, it can be seen that during the spring tide cycle, there are two distinct high tides, one higher than the other. Is it possible that these lower high tides are the non-ordinary spring tides? There is probably no answer to this question. It is the same with the neap tides. There is a lower and a higher. Is it the higher that is the non-ordinary?

Attorney General v. Chambers only cited two equinox high tide events that would constitute the selection of non-ordinary tides, but as can be clearly seen that criteria does not necessarily hold valid. It was Justice Hale who suggested that the spring tides of the month, at the full and new moons, were not ordinary tides. It may well be that he was not trying to categorise the tides themselves but whether the high tide event would ordinarily or regularly cover the ground.

Any high tide event that was greater than the lowest high tide would cover the ground so the lower high tides, the neap tides, could be considered to usually, or ordinarily, cover the ground. Justice Hale also considered that ordinary tides were neap tides. This would make all neap tides ordinary tides. If this is the situation, then the Surveying and Spatial Information Regulation use of the term ordinary when associated with neap tides is redundant or inappropriate.

If on the other hand there are non-ordinary neap tides, then can they be identified? Which of the neap high tides in Figure 8 could be considered as not ordinary and could be excluded? The judges in *Attorney General v. Chambers* did consider such a proposition: "*the same that excludes the highest tides of the month excludes the lowest high tides for they happen as often as each other*." There are at least some that think the higher and lower tides need to be first excluded (Corkill, 2013).

A random level could be chosen, as was done for the spring tides, and the neap tides falling short could be excluded as being non-ordinary. The lowest high tide is -0.05 m and so a value

of zero might seem appropriate as the criteria. There are 8 tides lower than zero occurring during October and spread throughout February, March and April with considerably more tides close to zero spread throughout the year. Clearly the same arguments against making any exclusion based on such a level criteria for spring tides equally applies to neap tides. There is simply no justification for doing such.

Making any exclusion from high tide observations using the level in which the tide reached as a measure of ordinary would introduce a bias into the observation and as stated introduce an artificial barrier to the natural movement of the tide. It could also unintentionally lower or increase the height of the determined mean and take the surveyor away from the natural mean. Any tide event that is a natural result of the influences of the sun and moon could be considered as an ordinary tide and would have to be included in the observation set. Quite possibly only tides heavily influenced by extreme weather events such as during a flood could be considered as not being ordinary. Creating an artificial upper and lower barrier of the tidal range cannot be justified.

Although the *Attorney General v. Chambers* judges were trying to come to grips with an explanation of ordinary tides, it is not clear as to whether or not they intended to exclude the higher spring tides or indeed any tides. An interpretation of their deliberations is that they were more concerned with the reach of the tide that would ordinarily occur; that point on the foreshore which the tides would more often reach as not. They were debating what would be the limit of the Crown ownership of the foreshore or that part which would ordinarily be covered by the sea given the periodic influx of the high tides: "*It is true of the limit of the shore reached by these tides that it is for more frequently reached and covered by the tide than left uncovered by it.*" "*The Crown's right is limited to land which is for the most part not dry and maniorable.*" Although the judges seemingly became tangled in the issues between what the tide ordinarily covered and what was an ordinary tide, the two concepts may not necessarily be equated. One interpretation is that the extent of land, the limit of ownership, was the more important factor and not whether or not the tide that made that line was an ordinary tide. In the end the judges did not use the word ordinary, undoubtedly because it was "*a word of doubtful import*", and instead opted for a simpler line of demarcation; "*the medium high tide between the springs and neaps*". The use of the word ordinary in the Surveying Regulation definition may well be misdirection by those who tried to come to grips with this concept of ordinary tides when there was no need to.

In their deliberations the judges of did not simply leave the problem with just their final ruling but also tried to provide some insight as to what they considered would be a means of measuring this tidal event that would ordinary cover the ground. It is from these deliberations and the wording of the Surveying Regulation that the surveyor can consider what methods could be employed to fully satisfy the definition of mean high water.

3.3 Between or Including Spring and Neap Tides?

A key word in the Surveying and Spatial Information Regulation definition is "between". The use of "between" makes this definition distinctly different from the ISG manual definition. One interpretation of between is that it does not include the end points but includes everything else in between. For surveyors to comply with this interpretation, they must first exclude the spring and neap high tides (setting aside the ordinary part for the moment), then take the mean of all the high tides that are left.

The identification of the spring and neap tides is, as has been shown, problematic and may not be able to be achieved in any positive manner. It may only be possible to identify a high tide as being either characteristic of being a spring tide or characteristic of being a neap tide. Without a precise definition to identify a spring and neap tide, it is not possible to identify the high tides that need to be excluded in order to take the mean of those high tides left in between. It is quite probable that it might be impossible to satisfy the Surveying and Spatial Information Regulation definition of MHW if the surveyor must exclude the end points and only take the mean of those observations in between. Given a set of data such as that in Figure 8, which are the spring high tides and which are the neaps? Which tides need to be excluded and which tides should be included to get the mean?

One possible interpretation of spring and neap can provide an answer, of sort. Presume first that the spring tide is a singular event determined by the greatest range between consecutive high and low tides (see dictionary definitions) and that the neap tide is the converse, having the least tidal range. Then using tidal data from a tide gauge, in this instance Port Macquarie from 1 July 2003 to 30 June 2004, both the high and low tides can be identified and the difference, or range, between computed. The result, shown in Figure 7 for September-October 2003, is then readily interpreted to find the greatest and least ranges for each cycle and which high tide can be considered the spring and which the neap. The mean can then be taken of all the high tides between but not including the identified tides for the entire 12-month dataset.

The resulting MHW values for "between" spring and neap tides (Figure 9) show a remarkable similarity to the moon phase determination (Figure 3) with just as much variation, 0.32 m in this instance. There are other differences not immediately discernible. The maximum time between the moon phases was 8.6 days and the minimum 7.4 days. This means the moon phase determinations have between 17 and 12 high tides contributing to a mean. This compares to the between neap and spring tide range of 10.4 to 3.6 days or as much as 20 high tides or as low as 7 high tides (see also Table 1). With such differences it would be expected that there are significant differences between the two determinations. As the moon phase determination was not acceptable because of considerable variability, so too are these results. However, the results do satisfy to some extent the requirements of the Surveying and Spatial Information Regulation definition for MHW. There is still however an additional factor that must also be considered.

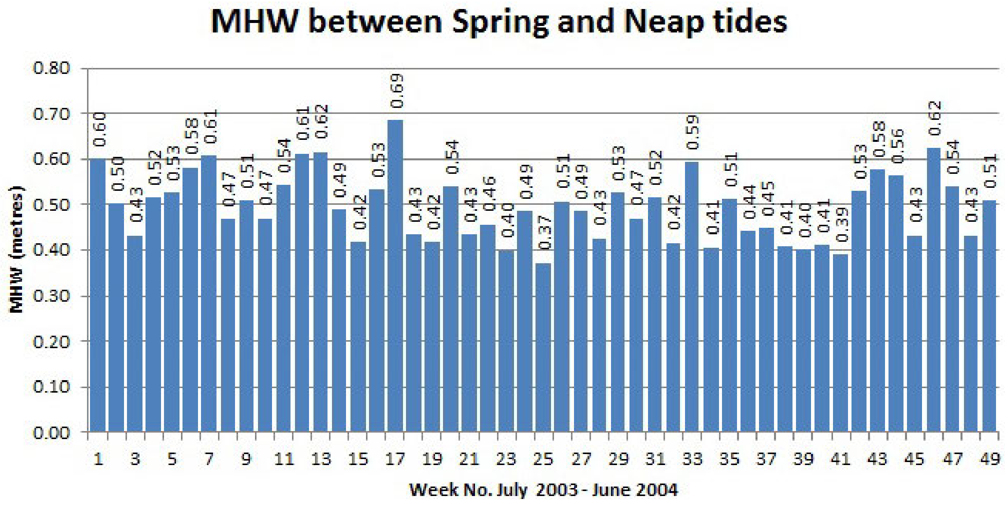

Figure 9: MHW between but excluding spring and neap tides at Port Macquarie in July 2003 – June 2004.

Now add in the uncertainty of ordinary. The above exercise excluded all the spring and neap tides but the definition requires only the non-ordinary tides. By virtue of the label it would mean that some spring and neap tides should be included. It is however not possible to identify which tides are the ordinary and which are the non-ordinary. Where does this leave the surveyor? Possibly close but not necessarily close enough to be fully compliant.

Another interpretation of between is that the mean tide, between the spring and neap, is a result of both. That is, it is the average of both the spring and the neap which exists as a mean in between. This interpretation however changes the practice to align more with the definition in the ISG manual and potentially taking the surveyor away from the legislation. Going back to the data that generated Figure 9, the spring and neap tides can be put back into the computation mix and a revised determination for MHW made.

The inclusion of spring and neap tides (Figure 10) looks almost identical to the excluded tides result of Figure 9. For the most part it is, as the variation of 0.28 m is similar, but careful examination reveals that the differences between the two results can be as much as ±0.04 m. This is a consequence of the inclusion of the end point high tides, the springs and neaps, which creates a different balance in the mean. The supposition examined earlier that exclusion of some tides could produce a bias in the result would appear to be well founded. It also stands to reason that the two methods are not interchangeable as they produce differing results and potentially different MHWM boundaries for the same period of observation.

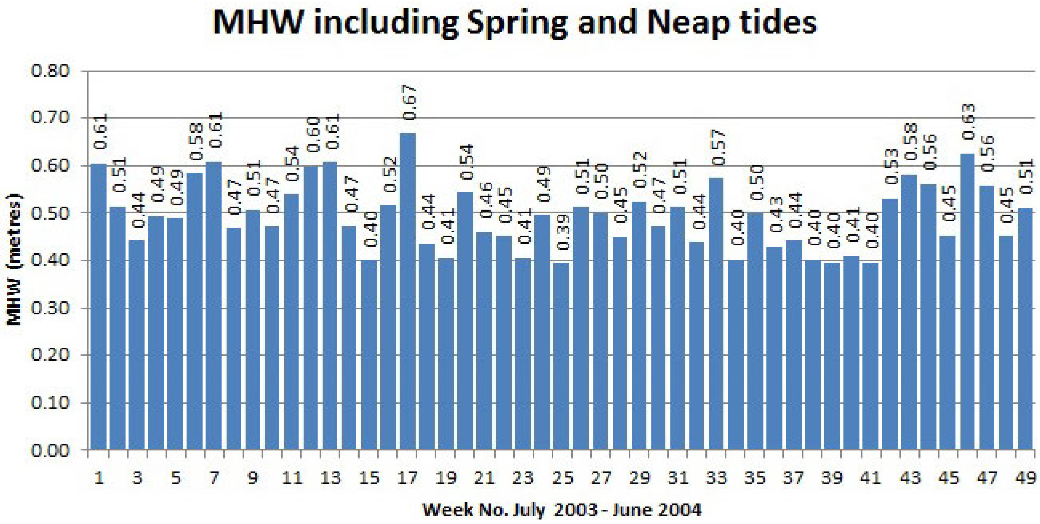

Figure 10: MHW including spring and neap tides at Port Macquarie in July 2003 – June 2004.

While the Surveying and Spatial Information Regulation requires the surveyor to subjectively exclude the non-ordinary tides, the ISG manual does not and instead requires the surveyor to consider all tides. The results of Figure 9 potentially satisfy the Surveying and Spatial Information Regulation and those of Figure 10 the ISG manual but there is a difference between the two in the results. If the non-ordinary tides cannot be identified, where does that leave the surveyor? The only possible answer is the ISG manual definition. This presumption would mean that it may be impossible to comply with the Surveying and Spatial Information Regulation definition as it requires exclusions which are not identifiable.

With the uncertainty and ambiguity of identifying particular tides for compliance with the Surveying and Spatial Information Regulation definition, how does the surveyor evolve a set of practice to determine the mean in mean high water?

4 MEAN HIGH WATER DEFINITION IN PRACTICE

The Surveying and Spatial Information Regulation definition provides inadequate guidance as to what practices would achieve a result that would comply with its requirements. It is up to the surveyor to make the most of what guidance there is. Some guidance can be gained from the deliberation of the judges in *Attorney General v. Chambers* who were seemingly attempting to describe a point on the foreshore where the high tides more often or not reached: "*What are then the lands which for the most part of the year are reached and covered by the tides? … the limit of the shore reached by these tides that is more frequently reached and covered by the tide than left uncovered by it. For about three days it is exceeded, and for about three days it is left short, and on one day it is reached. This point of the shore is about four days in every week*." Thus one practice method that the judges appear to have opened up to a surveyor is to observe every high tide for a week and find the height of the tide that would only cover the land four out of seven days. The three highest tide values for the week need to be excluded with the fourth highest being the desired criteria of MHW for that week.

In 2005 it was shown that the results from determining MHW by the height reached four days in seven varies considerably (Songberg, 2005). Making the comparison once again, but this time for the Port Macquarie data already utilised, the results from this method produce even wider results than those already discussed (Figure 11). Here the range between highest and lowest is 0.56 m and the largest of the methods considered, but more surprising is the higher trend in all the values. This method produces a mean for the whole year of 0.66 m whereas the other methods provide a yearly mean of 0.49 m. The criterion of four days in seven does not agree very well at all with other methods and consequently the judges may not have been as learned as they thought. This method can likely be discounted entirely.

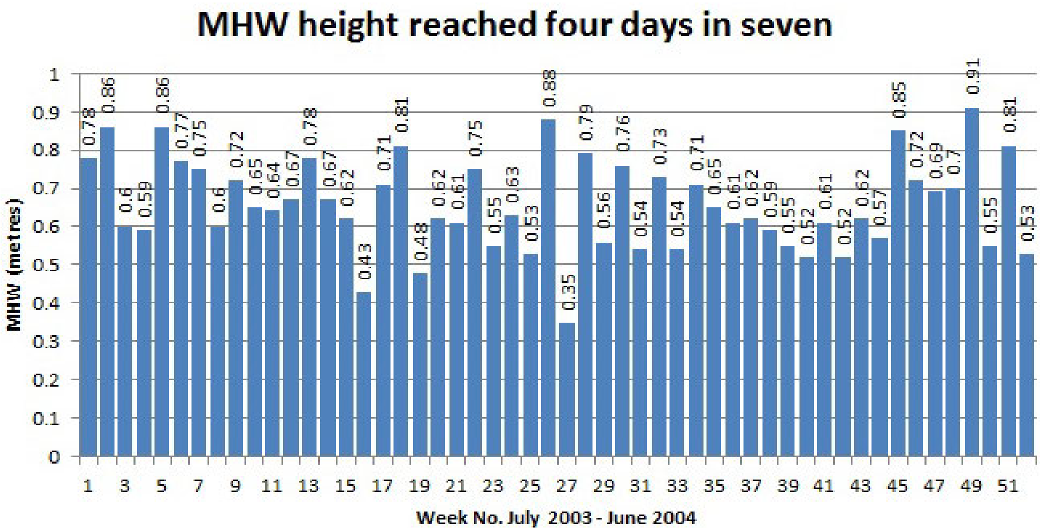

Figure 11: MHW as height reached 4 days in 7 days at Port Macquarie in July 2003 – June 2004.

The other approaches that were also considered by the judges in as to the limit of the sea or land were: "*The medium tides therefore of each quarter of the tidal period afford a criterion which we think may be best adopted … the average of these medium tides in each quarter of a lunar revolution during the year gives the limit … the limit indicating such land is the line of the medium high tide between the springs and neaps*." These deliberations can also be considered as plausible practice methods for determining mean high water and still be compliant with the judges' intent.

The results however are not necessarily the same. The measure utilising the moon phases and that of the line between spring and neap tides has already been shown. Although the results appear remarkably the same, they are not. Comparing the differing methods that were seemingly sanctioned by the judges of *Attorney General v. Chambers* as providing a measure of mean high water, it is soon found that the methods do not give the same result (Table 1). (It must again be stressed that the spring neap tide identification is only on the presumption that they are singular events and determined by the greatest range between consecutive tides is the spring and least range the neap. This presumption may not hold true.) The first three methods in Table 1 are those considered by the judges as providing a measure of the ordinary tidal event that usually covers the land. The fourth also compares the ISG possibility of a measure.

Table 1: MHW methods compared, data results from Port Macquarie in July 2003 – June 2004.

	Sample Interval				**Sample Quantity**		**Value**		
Method	**Max (days)**	**Min (days)**	**Ave (days)**	**Number intervals**	**Max (days)**	**Min (days)**	**Max (metres)**	**Min (metres)**	**Ave 1yr (metres)**
Moon phases (Fig. 3)	8.6	6.2	7.4	48	17	12	0.67	0.37	0.496
Betw. spring & neap (Fig. 9)	10.4	3.6	7.3	49	20	7	0.69	0.37	0.494
4 days in 7 (Fig. 11)			7.0	52	14	13	0.91	0.35	0.657
Incl. spring & neap (Fig. 10)	10.4	3.6	7.3	49	22	9	0.69	0.39	0.494

It is clear that there is no consistency with the resulting MHW from practice methods considered. The inconsistency is a consequence of the observation period being limited to the interval of the spring and neap tide cycle or an approximation to it (about 7 days). With results that vary over a 0.56 m range, depending on the method and time interval, the horizontal translation to the shoreline could result in differences in the tens of metres across a fairly flat foreshore. It is very doubtful that the judges in *Attorney General v. Chambers* envisaged such an ill-defined position to mark the boundary between the Crown and freehold foreshore. Community and property owner expectations of today would also not tolerate such a loose definition of the land. It is thus likely that none of these methods form an acceptable practice in determining mean high water.

The results shown have not included the identification of non-ordinary spring and neap tides so that none of the results will be fully compliant with the Surveying and Spatial Information Regulation. The closest that could be achieved is the “between” spring and neap dataset. This dataset excluded all spring and neap tides. Unfortunately the non-ordinary cannot be determined, so there is no way in which the surveyor can determine which is included and which is excluded in order to comply with the Regulation. This situation could possibly invalidate the Surveying and Spatial Information Regulation definition.

If the methods envisaged by the definitions and the judges of *Attorney General v. Chambers* are not appropriate in seeking an acceptable practice, the surveyor is left to follow historical procedures or other published guidelines. Of the guidelines that are available for the determination of MHW, the ISG manual ones are those that stand out. However, the practices recorded in the manual are synonymous with the manual’s definition of MHW and thus are not compliant with the requirements of the Surveying and Spatial Information Regulation as no exclusions are made. The ISG manual definition distinctly requires all tides to be considered. The ISG manual definition also goes away from the *Attorney General v. Chambers* expectations in that it adds an additional criterion, i.e. “*taken over a long period*”.

An examination of the ISG manual practices has shown that they fall considerably short of the expectations published (Songberg, 2005). A range of more than 0.2 m is quite probable between determinations and methods. Although not as bad as the 0.3 m result from the spring-neap cycle methods (or 0.6 m if the 4-days-in-7 method is considered valid), the ISG manual methods still do not provide consistency. What has not been considered and which now needs to be examined is what constitutes the "*taken over a long period*" part of the ISG definition? Do results change with variations in the length of the period?

Like the Surveying and Spatial Information Regulation, the ISG manual is not really helpful in defining a long period. It does, however, indicate in paragraph 22.5 (point 2) that "*observations extending over at least 12 months are necessary to obtain accurate results*". This statement would seem to suggest that observation sets shorter than 12 months are not acceptable. This would mean that the Surveying and Spatial Information Regulation spring to neap (7 days) and the manual's own lunar cycle (29 days) practice methods are both unacceptable. But is a 12-month period acceptable and does it provide any better result?

Manly Hydraulics Laboratory publishes tidal values for their tide gauges throughout the tidal reaches of NSW. One of these values is MHW determined over successive 12-month periods. It is these results that are today's mainstay of surveying MHWM boundary practice. But are they any better than the spring-neap cycle determinations?

Figure 12 shows the results from the Port Macquarie gauge over a 20-year period. As can be seen, there is a 0.12 m spread in the 12-month mean determinations. This at first seems a significant improvement over the 0.3 m spread of the spring-neap dataset. Port Macquarie, however, is only one of over 200 gauges and results vary considerably over the network. The spread in the 12-month determinations at any one gauge site vary from a low of 0.04 m to a high of 0.33 m. Clearly, a 12-month observation length provides little better results than a 7-day observation length.

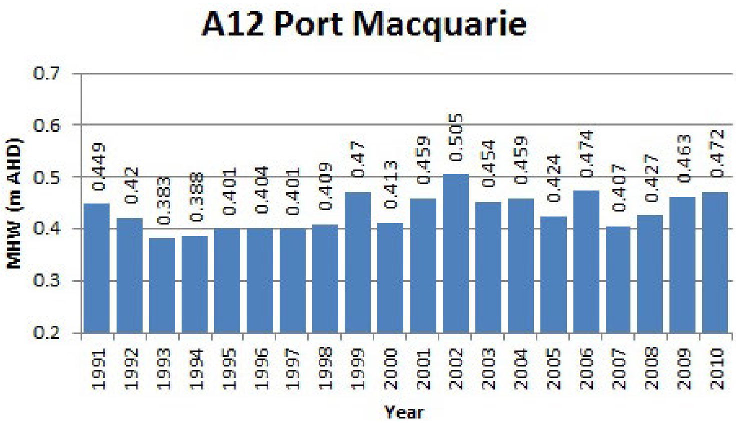

Figure 12: Yearly MHW values at Port Macquarie 1991-2010.

For Port Macquarie, extending the observation length to 2 years reduces the spread between highest and lowest means only marginally to 0.1 m. As the observation length increases, the spread does reduce to 0.07 m for a 5-year period and to 0.04 m for a 10-year period. Determinations of the mean from all sites exhibited a similar reduction in the spread between maximum and minimum datasets as the observation length increases. From Figure 13 it can be seen that the value of MHW seems to settle down after meaning an accumulation of about 6 months observing every high tide event. For a surveyor to accumulate such a dataset, they would need to observe somewhere in the order of 350 high tide events. Such a scenario is not

likely to occur unless a temporary tide gauge is installed to perform the observations. This also seems to suggest that at least a 6-month observation period is required for any meaningful answers to be achieved, not a 12-month period as indicated in the ISG manual.

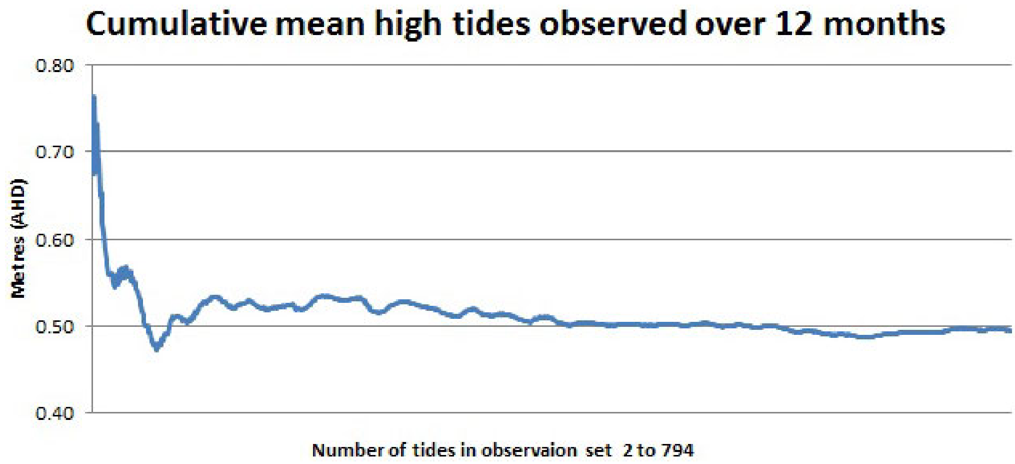

Figure 13: Cumulative mean high tides at Port Macquarie in July 2003 – June 2004.

The latest MHW values published by Manly Hydraulics Laboratory (Couriel et al., 2012) are a result of the mean of individual 12-month means over 2 to 20 intervals. Computation of cumulative means for records of 18 to 20 datasets found that a similar settling of the mean does not necessarily occur. Some sites seem to exhibit such a settling but some do not. Many site records are for considerably shorter periods. As a consequence, the MHW value over the period could still be subject to significant variability. As a result, the suitable length of "*taken over a long period*" remains uncertain. A lot will depend on the site conditions and particularly the variability of the tides.

5 SURVEYING REGULATION VS. SURVEYING PRACTICE

In today's surveying of the MHWM boundary, there is almost a total reliance on the published values of MHW from Manly Hydraulics Laboratory. Surveyors adopt the Australian Height Datum (AHD) level of MHW as published and simply level from the nearest coordinated marks out along the foreshore to establish the MHWM boundary.

In determining MHW values, Manly Hydraulics Laboratory does not undertake to mean the high tide values (Blume, 1995) nor do they selectively mean the tides between the ordinary spring and neap. Instead the MHW value is, as with all the other values, a result of a harmonic analysis of the entire dataset from the various tide gauges ranging from 12-month to 20-year periods. Only data that has been influenced by fresh water inflows has been removed (Couriel et al., 2012). The resultant MHW values are not a direct result of high tide observations but rather the result of a mathematical model. The practice used by surveyors that has evolved over time is thus unique unto itself and in theory would not meet with the regulatory requirements within the Surveying and Spatial Information Regulation for a MHWM boundary definition.

From contemplations of the definition of mean high water and possible practice methods that could relate to the definitions, what then is the mean of mean high water? For the data examined at Port Macquarie, there are the following possible determinations of the mean:

- From between the spring-neap intervals: 0.37 m to 0.69 m.
- High tide observations over a 12-month period (01/07/2003 – 30/06/2004): 0.494 m.
- Manly Hydraulics Laboratory harmonic analysis (01/07/2003 – 30/06/2004): 0.459 m.
- 20-year average of yearly harmonic analysis (01/07/1990 – 30/06/2010): 0.434 m.
- Year-to-year harmonic analysis determination: 0.383 m to 0.505 m.

None of these values fully satisfies the legislative requirements of the Surveying and Spatial Information Regulation as a value of the mean in MHW. The spring-neap means provide the closest possible answer to the Surveying and Spatial Information Regulation definition, and the 12-month mean of all high tides satisfies the ISG manual definition. The Manly Hydraulics Laboratory values for the mean unfortunately do not conform to either definition as they are not a direct result of the mean of high tide observations. What then is the true mean of mean high water? Is the mean the answer?

There is yet one more aspect to be examined: sea level rise. None of the values determined so far take into account sea level rise. Is there an effect and should it be included as part of the MHWM boundary deliberation?

6 SEA LEVEL RISE AND MHW

There are possibly still some sceptics out and about that do not believe that the sea is rising, just as there are sceptics that say there is no climate change. CSIRO and Manly Hydraulics Laboratory both fully recognise that sea level rise has occurred and is occurring. Over the last two decades, CSIRO provides a trend value of global sea level rise in the order of 3.2 mm/yr (CSRIO, 2014). It should be noted that CSIRO does stress that sea level rise is not uniform throughout the globe.

Manly Hydraulics Laboratory on the other hand provides a value of 0.94 mm/yr taken from data out of Fort Denison 1914-2004 (Couriel et al., 2012). However, the overall trend is a result of periods of apparently static mean sea level and other periods of apparent rapid rise. The two decades (1990-2010) covering the data analysis was a period of sustained El Nino which caused a depressing of sea levels and a flat trend in sea level rise. As a consequence, no adjustment of the data was made to consider sea level rise (Couriel et al., 2012).

The Fort Denison MHW values were published in 1994 (Ireland, 1994) and later extended to 2002. The chart of these values (Figure 14) provides some insight into what is occurring in respect to sea level rise, showing a linear rising trend of 0.8 mm/yr. Figure 14 also gives further insight into just how variable mean high water actually is. It is this variability that causes such a wide range in mean values (Songberg, 2005). But how much of this variability is influenced by sea level rise?

Earlier, the length of "*taken over a long period*" was considered. It was shown that the mean tended to settle down after about 6 months of high tide observations (Figure 13). The same tendency was not observed or was inconclusive in calculating the cumulative means for the 20-year values from Manly Hydraulics Laboratory (Couriel et al., 2012).

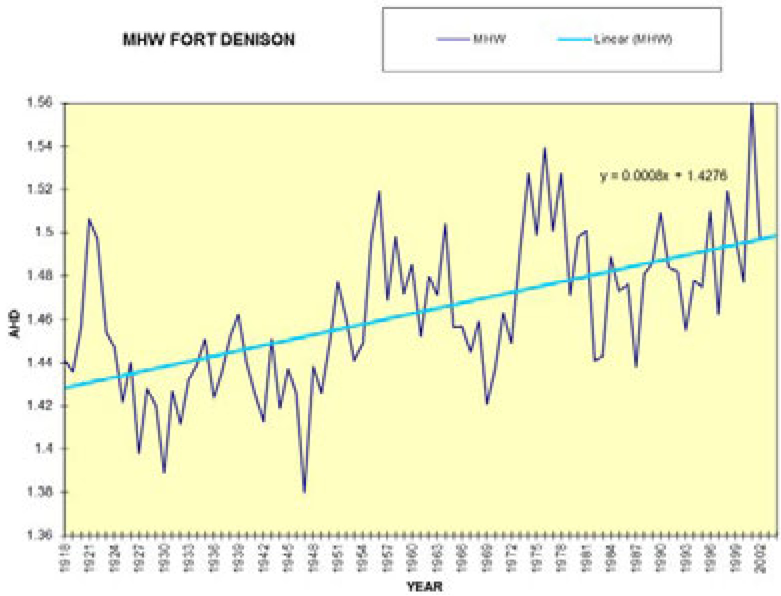

Figure 14: MHW values at Fort Denison 1918-2002.

Utilising the Fort Denison data to calculate an accumulative mean, there is also an apparent settling of the mean after about 20 years, or 20 observations, which continues for about another 15 years or observations. After this initial "*taken over a long period*" time, Figure 15 shows that the mean is then influenced by a rising sea level. The cumulative mean continues to rise as sea level rises. As a consequence of sea level changes, there does appear to be a limit to how long a period over which a mean can be taken before the sea level changes has an influence. The ISG manual definition now has a time limit.

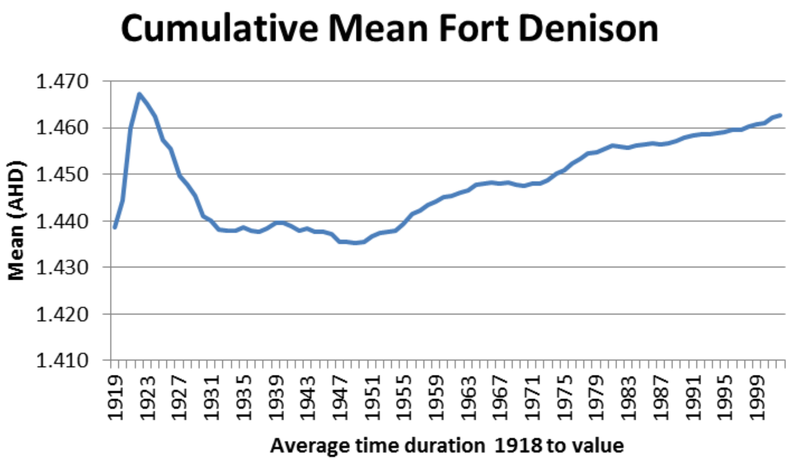

Figure 15: Cumulative mean values at Fort Denison 1918-2002.

But what of the 20-year records that are published by Manly Hydraulics Laboratory and predominantly used by surveyors? Is the 20-year mean (setting aside its non-compliance with the Surveying and Spatial Information Regulation) an acceptable period over which to take a mean value or should sea level rise be part of the consideration? Because of the various cycles affecting the moon and thus the tides, a 19-year period is considered as the most appropriate to establish the various tidal constants. The Fort Denison cumulative mean data suggest that it is only just long enough to determine the mean but is it long enough to show sea level rise?

Couriel et al. (2012) show around 200 sets of results of the harmonic analysis of the tide gauge readings. Excluding gauges that were set in non-tidal lakes such as Narrabeen Lagoon and those with only limited datasets, there are still at least 150 sets that can be interrogated to see if there is any tendency to show a trend for sea level rise. Creating charts for each dataset and plotting a linear trend through each, the results disclosed were a little mixed. The MHW values for Port Macquarie (Figure 16) show a rising linear trend of nearly 3 mm/yr. The data for Sydney, in this instance from HMS Penguin in Hunters Bay on Middle Head, shows only 1 mm/yr rising trend (consistent with the longer term Fort Denison trend).

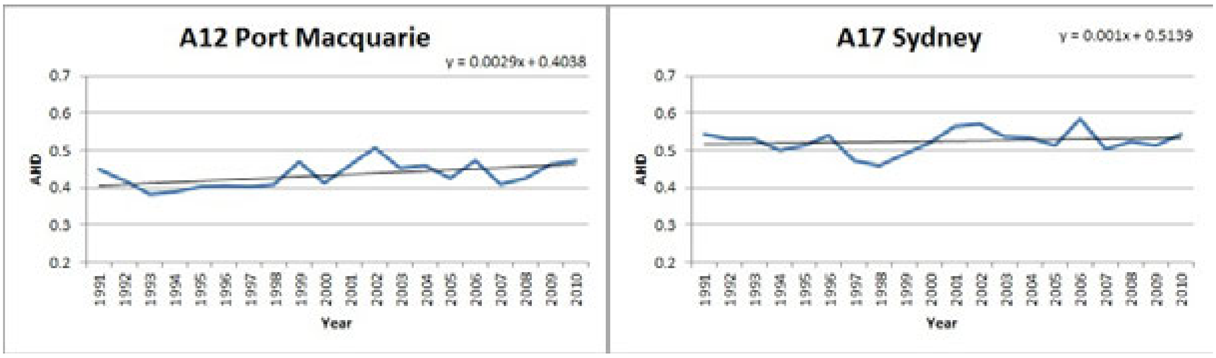

Figure 16: MHW trends over 20-year period (1991-2010) at Port Macquarie and Sydney.

However, not all tide gauge sites exhibited a rising trend. There were some that exhibited a negative or falling sea level trend. The 16 coastal gauge sites along the NSW coast provided a mixed result (Figure 17). The figures range from +4.7 mm/yr at Tweed Heads to -4.8 mm/yr at Crookhaven Heads. The overall analysis with gauges running up within the estuaries showed a similar mixed result (Figure 18).

A wide range of results was found from a -7 mm/yr to a staggering +23 mm/yr, suggesting that either the dataset is too short or there could be stability issues with the gauge. Overall, the results show a distinct tendency for the majority of datasets to show a positive linear trend indicating that sea level rise is most likely to be evident within the 20-year datasets. The average linear gradient suggests that for NSW there is a sea level rise of around 3 mm/yr. This result would mean that the sea level rise for NSW was generally within the quantum determined by CSIRO for the last two decades. It also indicates that the supposed flat trend in the sea level changes (Couriel et al., 2012) over the last two decades may not be as flat as first considered.

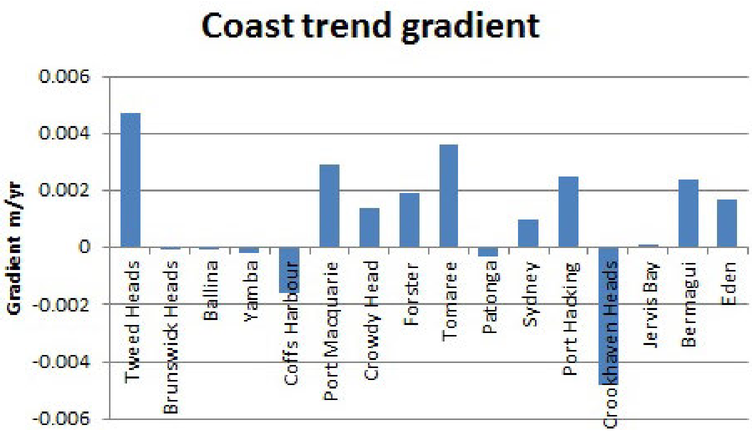

Figure 17: Linear sea level trends along the NSW coast (in metres).

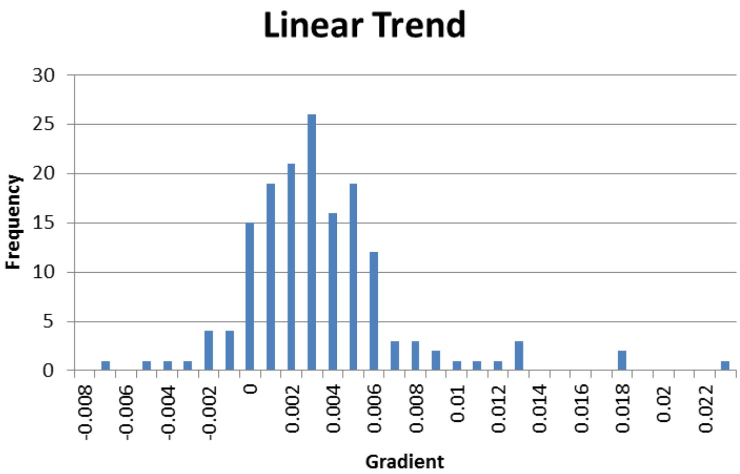

Figure 18: Linear trend histogram for all results.

As not all datasets included 20 groups, consideration was also given to find any correlation between gradients and the number of datasets. No correlation could be found. Within one standard deviation of the average trend gradient there were datasets with a count as low as 5. Outside the one standard deviation were datasets with counts of 18, 19 and 20 (Figure 19). Once again the result may not be a result of the data length but rather the stability of the gauge or even an issue with the AHD relationship. There was however a tendency for the smaller datasets to exhibit a higher grade and the larger datasets a lower grade.

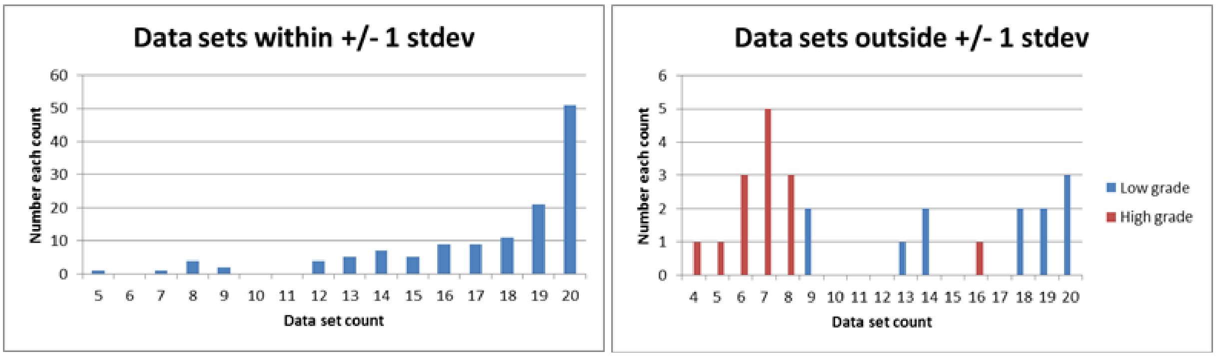

Figure 19: Count of datasets within and outside one standard deviation of the mean trend gradient.

With the distinct possibility that sea level rise is evident within the data used by surveyors, what implications does that have for the mean? The datasets used for this paper, and still commonly used by surveyors, are now 5 years out of date. If sea level rise has an effect on the data, then the mean derived from that data may not be acceptable. The mean may not be the answer. Expanding that dataset may also not be the answer, as sea level rise will continually affect the mean, as was shown in Figure 15. What can the surveyor do?

Surveyors could go against long accepted practice and adopt a 7-day tide observation period to determine a mean that is likely to be more compliant with the Surveying and Spatial Information Regulation, but the results would wildly fluctuate and would be unacceptable in today's expectations. The other option is for the surveyor to take into consideration sea level rise and account for it. The implication this has on the 1990-2010 datasets is that the value of MHW would need to be adjusted for sea level rise, not at the time of the data but for the time of the MHWM observation.

If that observation were today in 2015, what is the likely affect? Utilising only the 1990-2010 datasets and the derived linear trends, a predicted difference between the mean value and the trend value can be determined (Figure 20). The consequence is that there is likely to be around 20-60 mm difference in MHW between that published and that likely given sea level rise. If a surveyor were to use the published value then they would be too low in their estimate and have the boundary too far seaward. As time increases, the difference between the mean and the trend will become greater. Even if the survey were to have been conducted at 2010, the value of MHW used by the surveyor is likely to have been too low by about 10-40 mm.

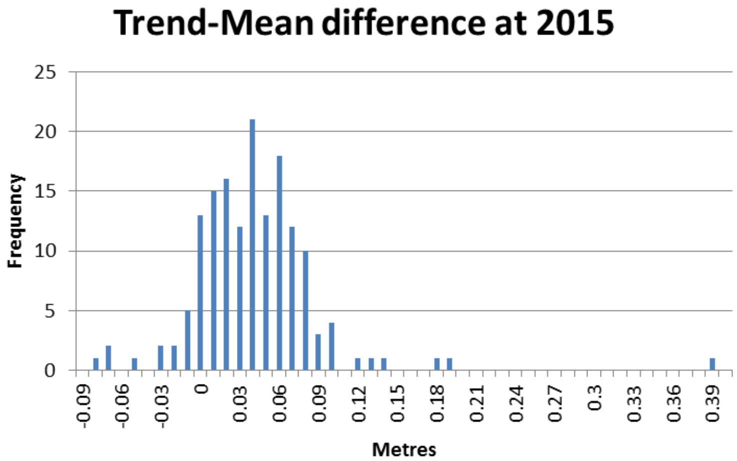

Figure 20: Difference between trend MHW and mean for 1990-2010 at 2015.

So is the mean the answer? If sea level rise is a fact and should be considered in determining MHWM boundaries, then it is likely that the mean will not be the answer. For many years the value of MHW adopted in Sydney remained static (Blume, 1995) but more recently it was felt that the value be periodically revised using a period of 5 years (Ireland, 1994). If the data examined is any indication, this 5-year increment between adjustments may not be an appropriate mechanism for dealing with sea level rise. As the published figures are always running behind the time of survey, in some cases years, more likely a trend, utilising the mean would provide a better realisation of MHW. It would also provide limited forward projection from the time of publication to the time of survey.

7 CONCLUDING REMARKS

Surveyors today would be surprised to learn that their MHWM definitions do not comply with the Surveying and Spatial Information Regulation 2012 and have not done so for a long time. Close examination of the definition of MHW in the Regulation and the data used by surveyors to firstly determine the MHW value, which is subsequently used to undertake the boundary definition, supports the belief. Surveyors would be further dismayed to learn that the definition in the Regulation cannot be complied with. The problem is in the wording of the definition. If strict adherence to the wording of the legislation is required for compliance, then the uncertainty in identifying the particular features of the definition such as spring tides, neap tides, non-ordinary tides and the need to exclude equally uncertain sets of data make compliance impossible. Data used by surveyors today in establishing the mean in MHW is not a numerical mean of high tides but rather the result of mathematical analysis over the entire tidal range and as a consequence does not comply with the specific requirements of the Regulation. References to spring or neap tides and the range in between must be removed from the definition and a new definition formulated that complements current survey practice.

Long-term data shows that sea level rise is a factor, and if the mean is taken over too lengthy a period, then sea level rise will have an influence. If not taken into account, then the resultant mean will be too low. Taking the mean over shorter periods, however, leads to wildly fluctuating results, a factor which would not be tolerated in today's need for more precise property definitions. The length of the period after which the mean becomes unstable as a result of sea level rise is somewhat uncertain. With only 20-year datasets available for examination, the results, although supportive of the concept, were not fully conclusive. Expansion of the data and more reaching analysis is required before a fully definitive result can be produced. However, from what has already been examined, the data does support the concept that the mean of mean high water is not as reliable an answer as was previously considered and that a new definition that will agree with current survey practice and take into account sea level rise is required for the determination of tidal riparian boundaries. Early data examination suggests that criteria utilising the trend in sea level rise coupled with mean high tide determinations from tide gauge harmonic analysis might be the way forward.

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Spatial Information Methods used at Southern Cross University's National Marine Science Centre

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ABSTRACT

Over the last two decades there has been increasing recognition that oceanography and fisheries sciences and related marine areas are nearly all manifested in the spatio-temporal domain. Geographic Information Systems (GIS), remote sensing and mapping, i.e. the natural framework for spatial data handling, are now recognised as powerful tools with useful applications in marine sciences, providing necessary information to decision makers to manage marine resources. This presentation will highlight some of the spatial information methods used at Southern Cross University's National Marine Science Centre to assist with marine science research. Delegates will visit the National Marine Science Centre and the accompanying Solitary Islands Aquarium to gain insight into the local marine environment and the research being conducted at the centre. Delegates will watch a short documentary, explore the aquarium and participate in a guided tour of the research facility. An additional presentation will provide particular insight into the spatial information methods utilised by marine researchers.

KEYWORDS: *GIS, remote sensing, mapping, marine science.*

Bearing Fruit: 20 Years of Property Information Management

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ABSTRACT

The then Department of Education, Properties Directorate resolved in 1992 to undertake the development and implementation of an IT system to manage all 2,400 of their school campuses. In 1994, OMNILINK was originally a sub-contractor to DIGITAL then Compaq on the first and second rounds of initial development of the Asset Management System (AMS), involved in the data management. The data capture was managed by the former Public Works Department and undertaken by contractors, of which OMNILINK's sister company Lockley Garvin & Partners was one. OMNILINK was contracted to prepare the specification for data capture, which is still largely used. Since the initial development, OMNILINK has been incessantly engaged by the Department to provide data management and system maintenance services and has also provided further development of AMS. This paper outlines the processes involved in data capture as well as the functionality of the system. It also provides a brief description of a cut-down version of the system called AssetWhere™ and its associated software, which OMNILINK developed and implemented for use in organisations with large property holdings.

KEYWORDS: *Property management, data capture, system development.*

1 INTRODUCTION

With Building Information Modelling (BIM) receiving great attention in the last few years, especially in relation to 'smart cities', the role of the surveyor has not had a great deal of attention. With modern technology such as terrestrial laser scanners, hand-held scanners, LiDAR, UAVs and side-view aerial photography, the surveyor will play an increasingly important part in the initial and ongoing data capture. Surveyors need to be aware that there are great opportunities to contribute to these important databases and be prepared for these.

This paper provides a case study of a data capture program that populated a large property asset management system (another name for BIM), the system itself and smaller systems. A discussion regarding asset information value to highlight what value exists in the surveyor's role with these programs follows.

2 NSW EDUCATION

The Property Directorate of the Department of Education & Communities (DEC) is responsible for over 3,500 campuses of primary and high schools and TAFE colleges throughout New South Wales (NSW). Just the built component of these properties has been

valued at $28 billion, and an assumption to double this figure to allow for land values would not be unreasonable. So the total value of the Department's properties in 2014 can be assumed to be in the vicinity of $60 billion.

Prior to 1992, the records for all these properties were paper-based and held in various places. Under the sponsorship of the then Deputy Director-General, Jan McClelland, a proposal was agreed upon to bring these paper-based records into a functional digital system. Planning commenced for writing specifications for the system. Prior to the NSW resolution, the Victorian Department had devised, produced and installed their Schools Asset Management System (SAMS) and NSW reviewed this in their investigations. The functionality of SAMS was limited, and the NSW plan was to extend this. A modular approach was formulated, representing the various responsibilities of the Directorate. Further detail on these will be provided later in this paper.

Proper management of real property assets requires knowledge of the location of these assets, what physical items have been built upon land and their relationship with other sites, within their own site and the site's relationship with its community, transport and other facilities and government services. Best practice required that representations of these items and their relationships together with other related information such as demography, student and staff locations and transport schedules be recorded and stored in a geospatially related database with a graphical user interface. Geographic Information Systems (GIS) integrated with relational database systems provided such a platform.

It is a testament to the planning and project management of the NSW Asset Management System (AMS) that the system has largely remained as planned in the past 20 years. OMNILINK has an ongoing role to manage all the school and TAFE data and to maintain the system with links to other DEC systems, system revisions and upgrades. The NSW Department of Education and Training received a Prime Minister's Award for Excellence in Government for the AMS system in 2000.

3 DATA CAPTURE

Data collection and integration applies to the initial and complete capture process as well as the ongoing data maintenance capture process due to property asset changes, removals and additions. This applies to spatial data that captures room, building, site and infrastructure layout. The required accuracy for dimensions and location of property assets will determine the data capture process.

DEC has outsourced the data capture to NSW Public Works who in turn use a panel of private sector survey firms to undertake the work either directly to them or through a building contractor who is constructing additions to or refurbishment of a school or TAFE or constructing a new campus. The choice when planning AMS was to capture data by survey or to utilise existing site and building drawings as the base plans. The decision to undertake the capture by survey was known to be a more expensive exercise but has yielded benefits in that all room and building areas are known to survey accuracy. This in turn allows more facility in planning additions and valuation of school property. Another major benefit of knowing room areas was the considerable saving in cleaning contracts. The Victorian Department, which captured data from architectural drawings (some of which were over 100 years old), saved the entire cost of the development and implementation of their system from these savings.

Prior to any capture being undertaken, Education and Public Works recognised the need to prepare data capture specifications. OMNILINK was engaged to undertake the consultancy to prepare this document and these have been in use with minimal fundamental revisions to date. The specifications were required to capture attribute and location data in an orderly manner for later input into the database, to provide accuracy standards for measurements, to specify data format and to assist with quality assurance. The specifications are very detailed and differentiate between point (small features), line and polygon (larger features) data for both buildings and site (Gillam, 1992).

Site features include the overall site layout, internal main access roads, car parks, sports facilities, assembly areas, bubblers, gas and fuel tanks, plants and sheds. Building features include major buildings, demountable buildings and rooms, which are defined as any space within a building including most cupboards, such as fire and electrical, ramps and corridors. Locational accuracy is generally 90% or 95% and dimensions are captured to 95% or 99% (Gillam, 1992).

Each building and each room within a building is allocated a unique identification number by the surveyor together with building and room usage and room name (if any). Other attributes to be captured include the materials of construction and treatment of walls, roofs, floors and ceilings, ceiling heights, security details, ventilation, heating, sink and toilet details and many others. The field capture data is processed by Public Works then transmitted to DEC for integration into AMS by OMNILINK.

The initial data capture commenced in1996 with high schools, then primary schools followed by small rural schools. Schools for the whole state were captured by a panel of eight surveying firms over a period of about one year. TAFE capture was undertaken later. The time taken is remarkable for its brevity, given the scale of the work, and Public Works is to be congratulated for its project management of this stage. Current capture is still undertaken by a panel of surveying firms that was enlarged at the time of the ‘Building the Education Revolution’.

4 AMS OVERWIEW AND DESCRIPTION

The initial system provided three basic modules (i.e. sites, facilities and spatial) based upon the migration of legacy systems that mainly dealt with assets from a purely financial and inventory basis. A geospatial view of assets was always seen as core to the system and has continued to be integrated to all business functions within the system. The system was later extended with three more modules that focussed on specific business units within the Directorate: planning, maintenance and demountables. AMS is now one of the largest spatially integrated systems within NSW Government and was also adopted for use within the NSW Emergency Information Coordination Unit for the management of a buildings database within the Sydney CBD.

The AMS project has evolved over the last 20 years from originally being focussed on strategic management requirements to now being a key integral part of many operational and tactical systems within the Properties Directorate. Many business functions of the Department are now solely conducted using AMS. Since its inception it is estimated that AMS has saved the Department in excess of $25 million and delivered operational efficiencies of equal or even greater magnitude.

AMS supports the asset management activities of the Properties Directorate within the DEC. The modules within AMS are:

- Sites – legal and administrative aspects of property asset management including acquisition, disposal, merging and valuation.
- Facilities – covers the details of building and rooms including elements which provide information on the asset that make up the rooms such as fixtures, external and internal materials, roof structures, etc.
- Spatial – a fully integrated set of information and functions that are available from within all AMS modules, i.e. the ability to always 'provide a picture'. Over 50 layers of spatial data are available within the system, showing details down to every room in every building on every site. Demountables and site-based structures are also shown within the system.
- Planning – provides support for the process of school and TAFE planning, including demographic analysis, catchment creation, enrolment data, entitlements and future projections.
- Maintenance – covers all building and plant maintenance information of all schools based on a series of contracts. This includes condition assessment data that is updated annually. It provides reporting tools to allow for analysis of past, current (actual vs. estimate) and future (budgeting) purposes.
- Demountables – provides support for both the management and operational tasks of demountable buildings that includes allocation of buildings based on school needs and entitlements, production of work orders for site preparation, transportation and delivery of buildings and is interfaced with the maintenance and planning modules. A spatial analysis tool provides shortest-path analysis for input into transport costs.
- Program Management, Minor Works – management of all aspects of capital works under $3 million. It covers the process from initial request through project specifications, construction and completion and has an extranet interface to NSW Finance & Services that has 300 users registered on AMS who can update the progress of any minor works project. This keeps all parties up to date and has removed the need to follow up for progress and status reports.
- AMS on the Web – provision of a simple Intranet browser-based system that provides each principal and TAFE property manager with a view of their school/college and includes maps/plans of all school assets.
- Numerous interfaces to other DEC business systems including financial and asset condition.

OMNILINK provides a data management and support role for the AMS system and is responsible for all data updates (spatial and aspatial). Data is updated at various frequencies including instantaneously (via database replication) to weekly, monthly, quarterly and annually. OMNILINK also provides a spatial data management role that includes brokering external datasets (e.g. ABS, PSMA, NSW LPI, NearMap).

4.1 Technical Data and Information

The AMS system is based on a Unix/Windows platform utilising Oracle and Esri ArcGIS Server as the core enabling technologies. The system supports the following data volumes listed against the key data items of sites (property), building, floor (level), room, elements.

- AMS Oracle database (40 GB).
- GNAF database (4 GB).
- Spatial database (40 GB).

- Sites 3,525 – includes pre-schools, schools, TAFE colleges and other DEC sites.
- 3,680 amenities.
- 23,283 buildings.
- 6,287 demountables.
- 382,894 rooms.
- 2,981,218 elements.
- Over 2,165 registered users.

The AMS spatial system is based on a platform of the NSW cadastre with all sites shown in their true geographic location. Other themes of spatial information include:

- Administrative DEC – layers showing DEC boundaries for operations.
- AMU regions.
- Catchment boundaries – primary, secondary, etc.
- TAFE institutes.
- Australian Bureau of Statistics – 5 census periods are held to allow for trend analysis.
- Electoral boundaries – LGA, state and federal.
- Postcodes.
- Suburbs and localities.
- Road centrelines – used for route determination of demountable buildings.
- BING aerial and road layers.
- Bushfire hazard zones.

5 THE DEC AMS SYSTEM

The home screen provides an overview and allows navigation to other modules and tasks (Figure 1).

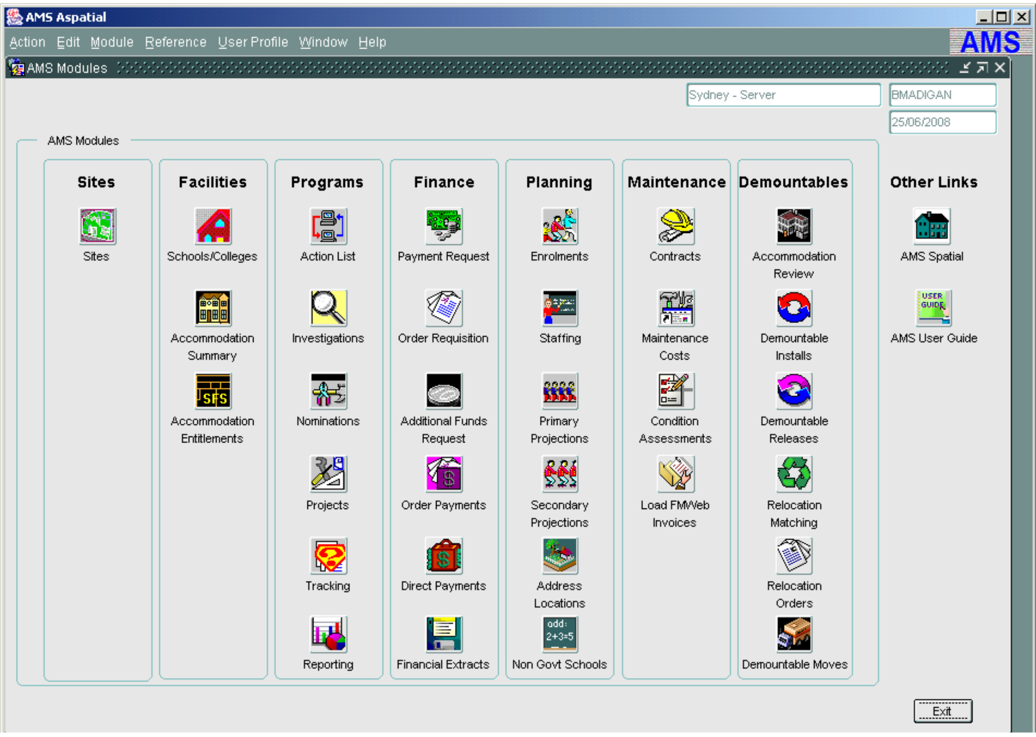

Figure 1: AMS home screen, allowing navigation to other modules and tasks.

5.1 Sites

The sites module allows input, query and reporting on sites, lots and leases (Figures 2-8). AMS incorporates legal and administrative aspects of property asset management. A legal description for every land parcel together with an outline of each landholding is located within the road centreline network for the state. The land planning zones for each DEC owned and adjacent parcels are also stored and displayed for every site. The location of the sites has a reasonable absolute accuracy together with facilities recorded and located on the site.

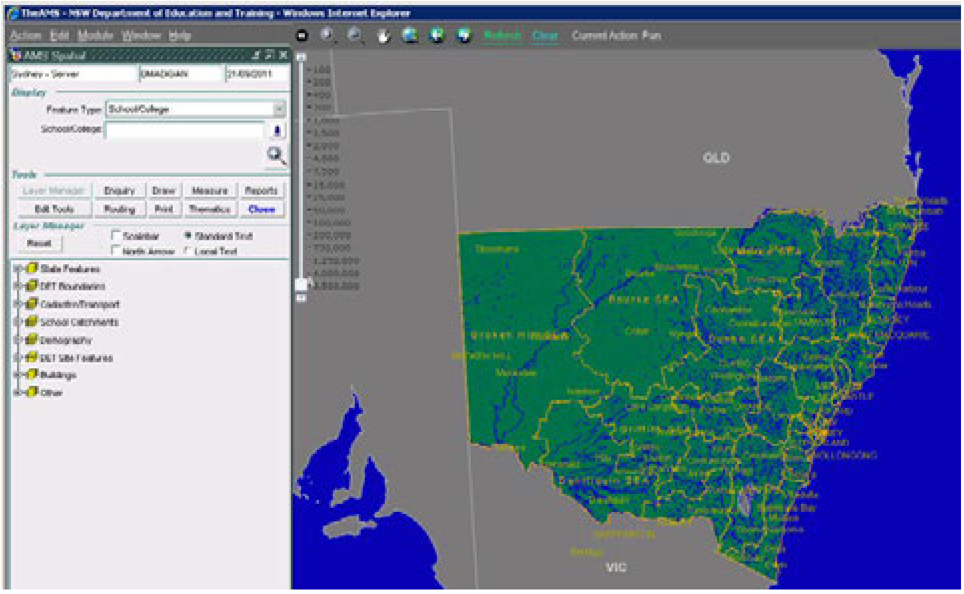

Figure 2: AMS spatial interface showing School Education Areas – the spatial entity used for many asset management functions, responsibilities and contracts.

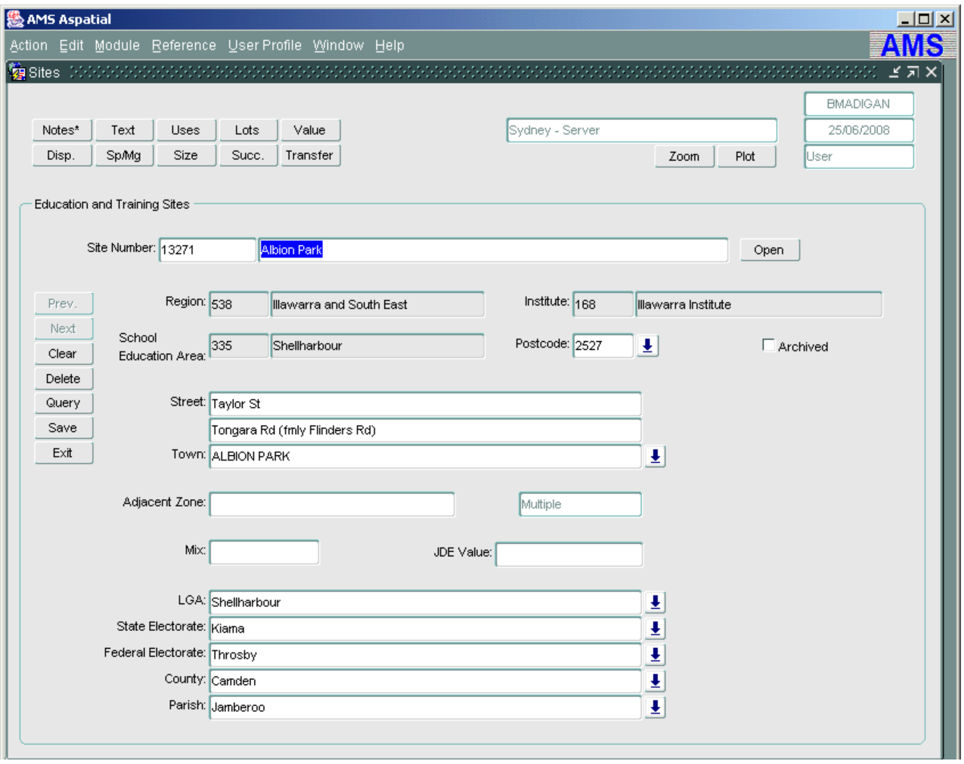

Figure 3: Sites module, a key module for spatial integration, providing the basis for land acquisition and disposal.

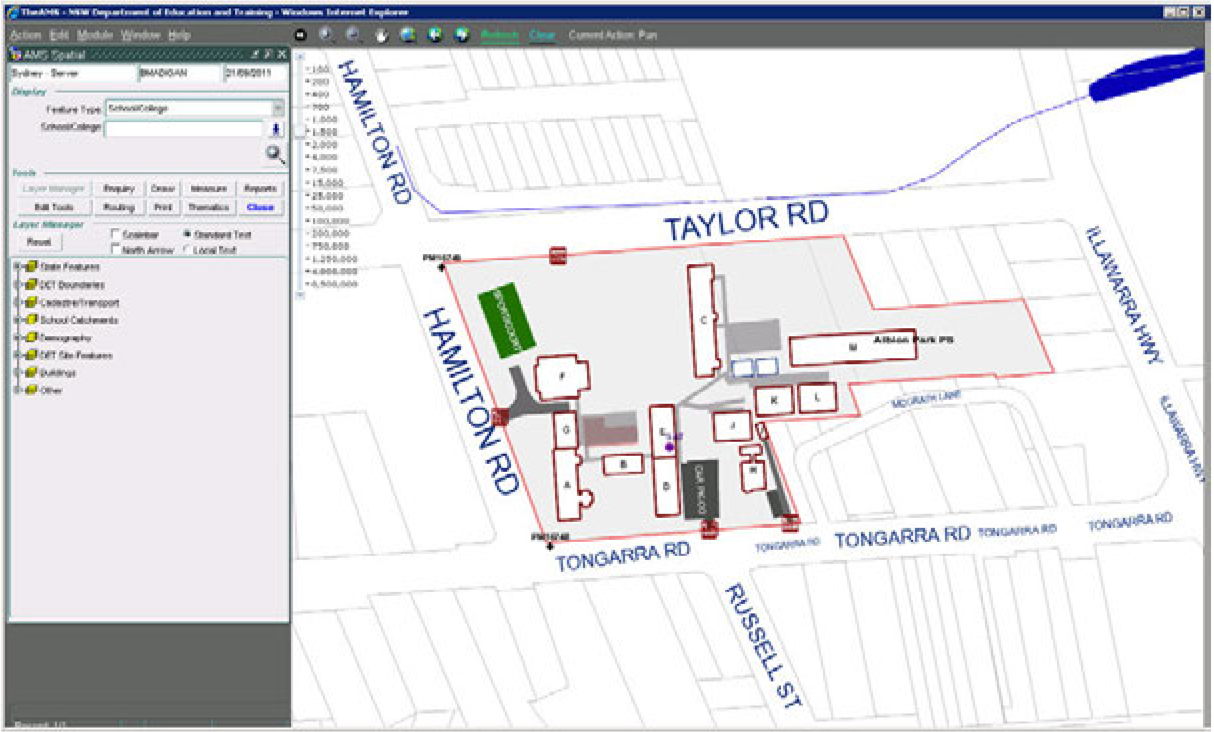

Figure 4: Detailed display of a school showing base layers of cadastre, site, buildings and sports facilities.

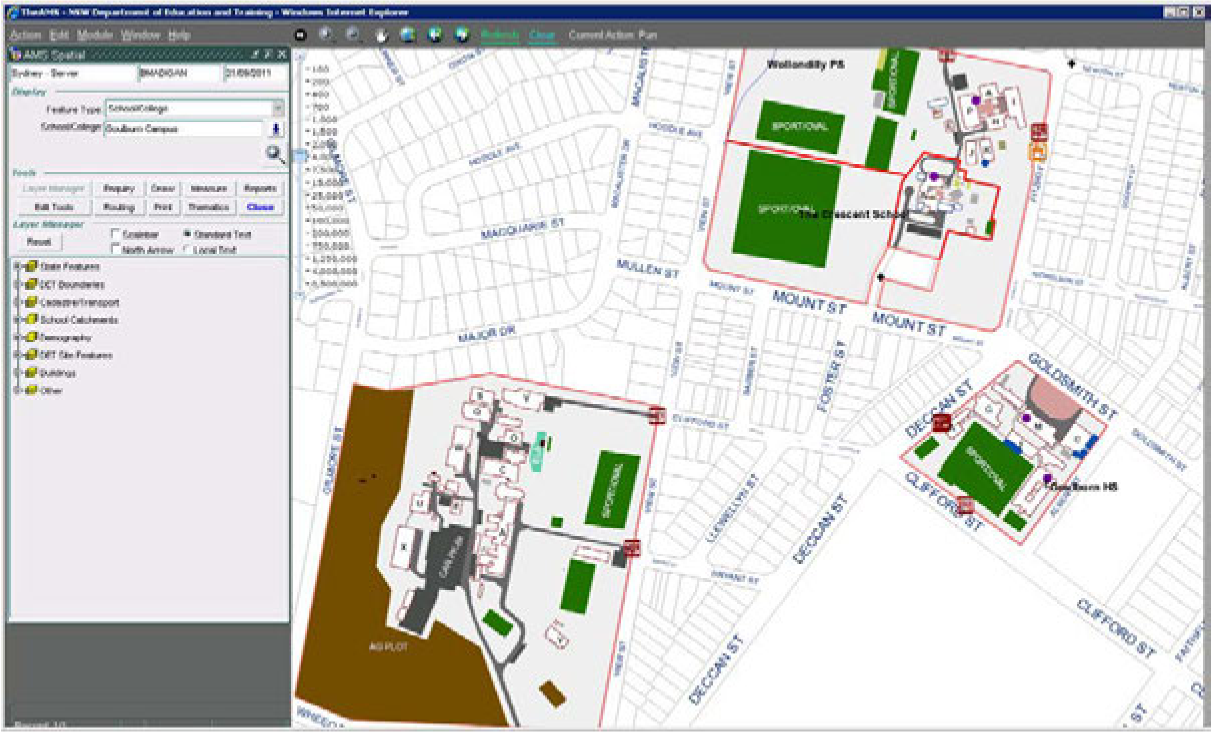

Figure 5: Display of a multi-site campus (Goulburn High School) including agricultural plot (northern site).

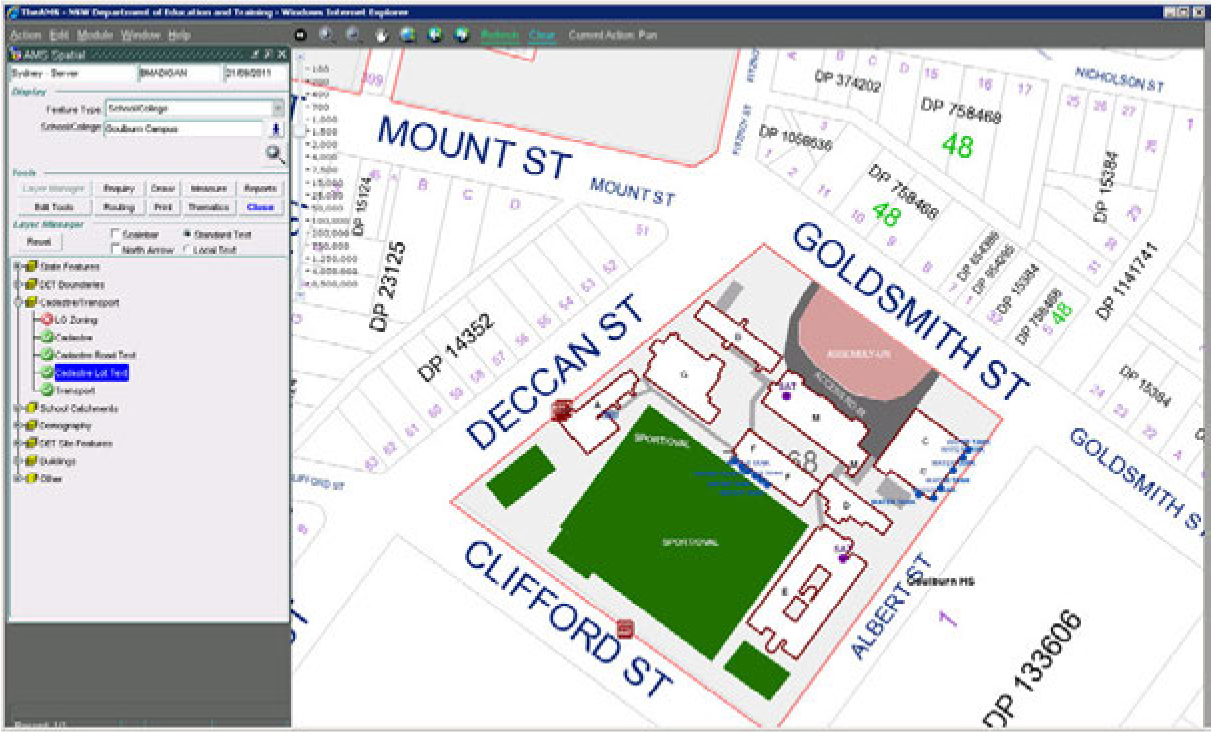

Figure 6: Detailed site plan of Goulburn High School.

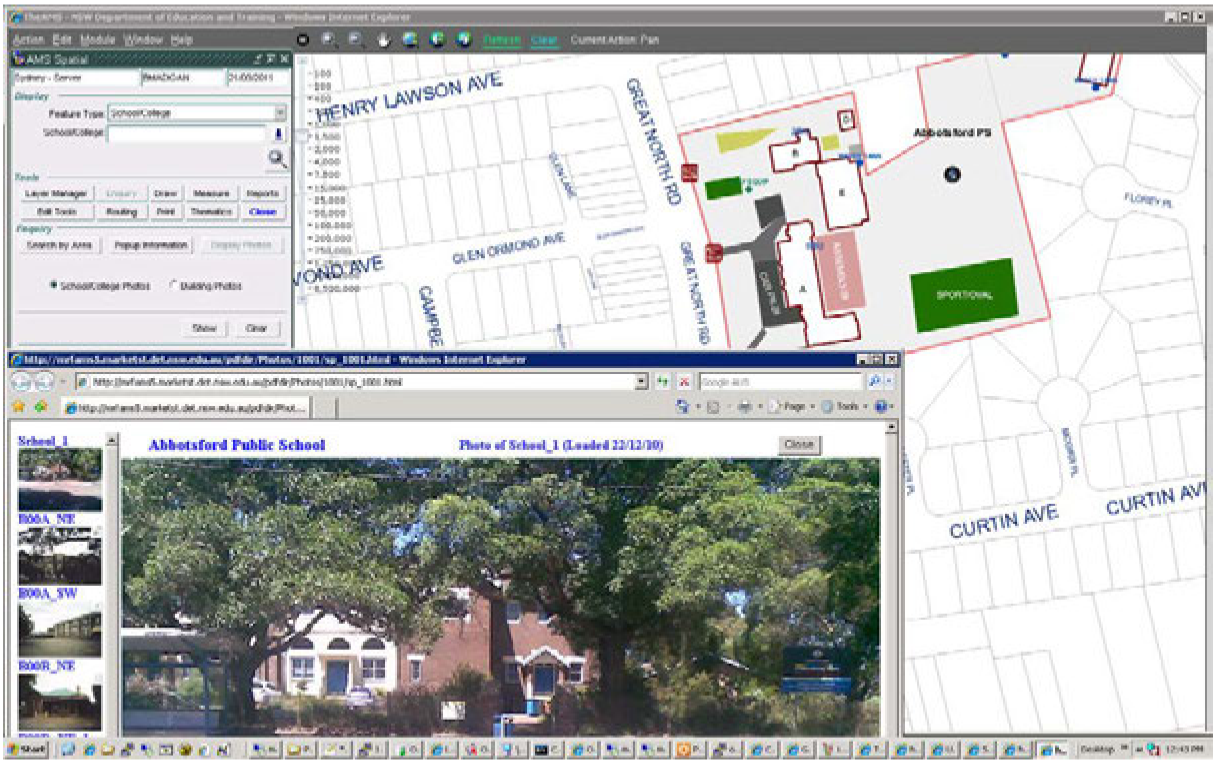

Figure 7: Photo tool linked to site features and locations.

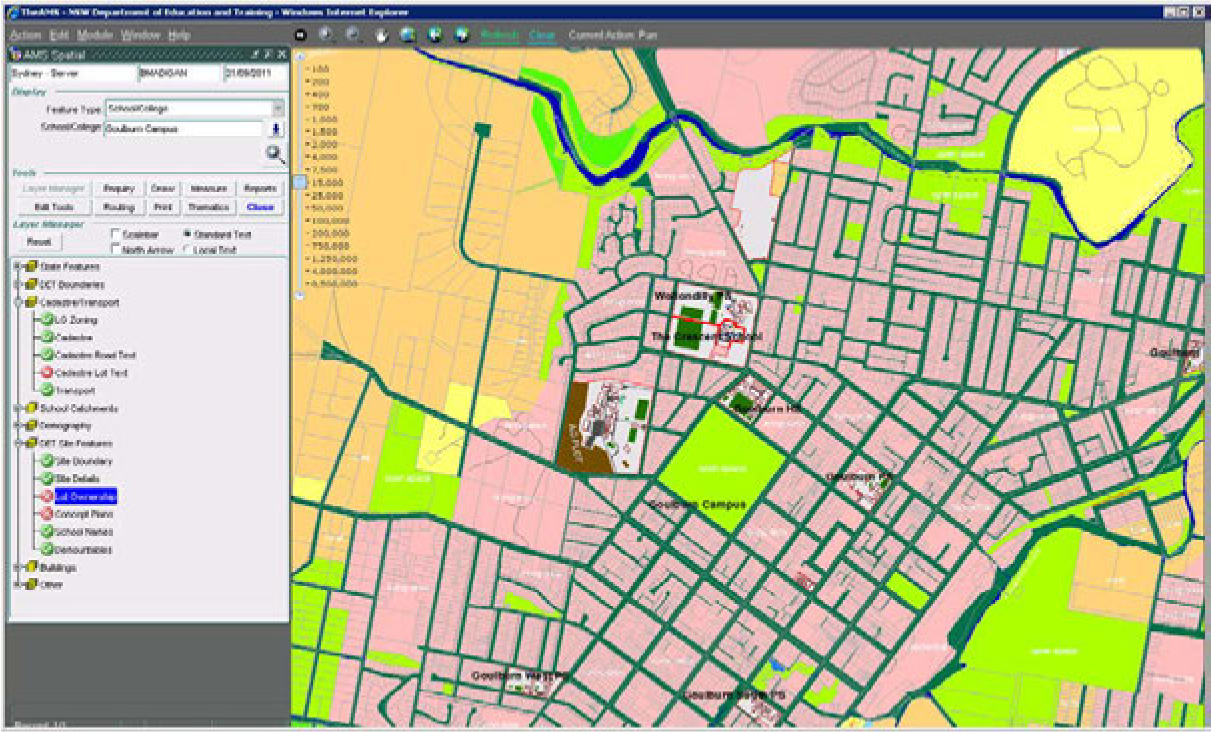

Figure 8: Using the cadastral data provided by NSW LPI, the Department can display attributes including tenure which can provide inputs to acquisition and site expansion activities.

5.2 Facilities

The facilities module records the details of buildings and rooms including elements which provide information on the asset that make up the rooms such as fixtures, external and internal materials and roof structures (Figures 9-12). All building footprints, site features such as plant and play equipment and floor plans are recorded for every building on every site. These details are kept current by data forwarded by the school or by surveying consultants on a regular basis. This AMS module allows input, query and reporting on facilities (in DEC's case, schools and TAFEs).

DEC classes all entities relevant to each facility as 'components'. The current component classes are buildings, rooms, demountables and amenities. Amenity is a broad classification ranging from sheds to playgrounds. The attributes of these are classed as 'elements' and include details such as internal materials and fittings. In addition to physical assets, the facilities module also encompasses functionality of each facility. This includes enrolment details (past, present and anticipated). AMS was initially primarily an 'asset management system', so the system groups components by functionality for reporting and spatial display purposes. Another facet of functionality can be capacity. This is approached in terms of both 'entitlement' (i.e. how much each facility has been budgeted or resourced) and 'capacity' (i.e. how much each component has been designed to facilitate).

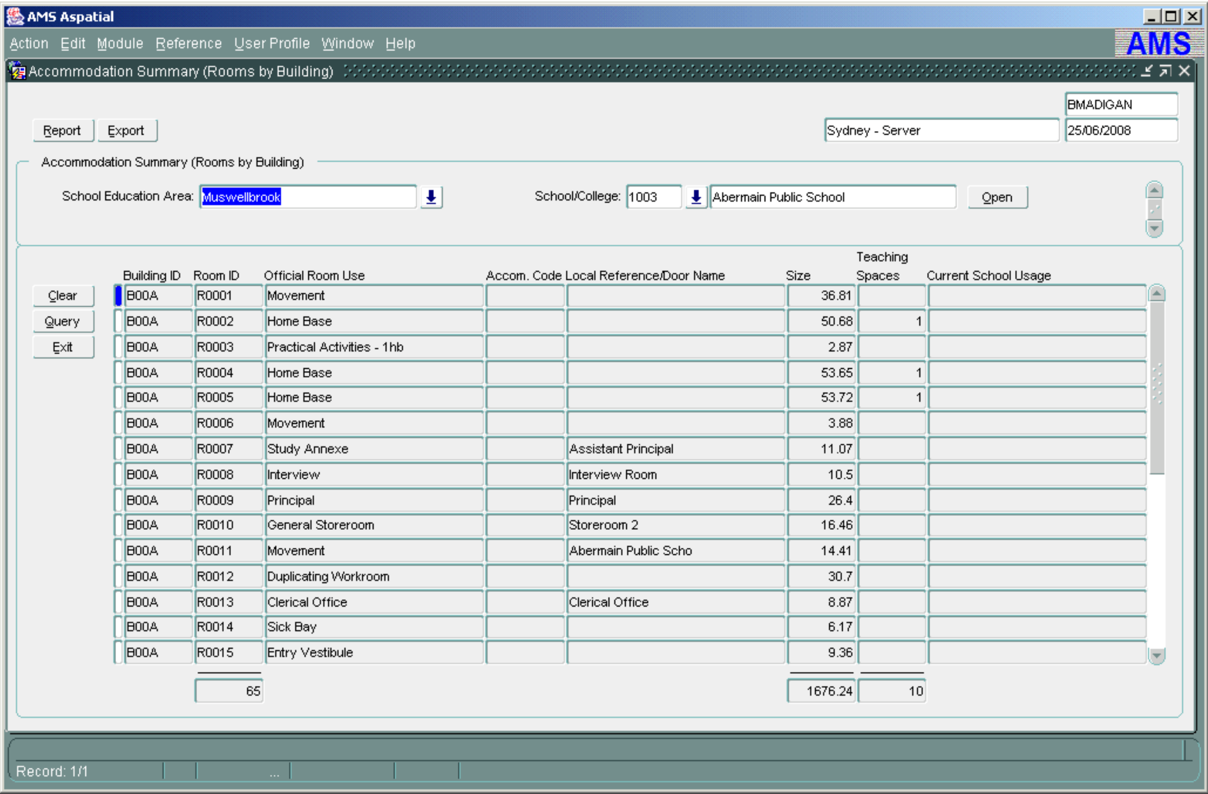

Figure 9: Facilities module showing room use, size and indicating which areas are for teaching – key input to entitlement calculations in the planning module.

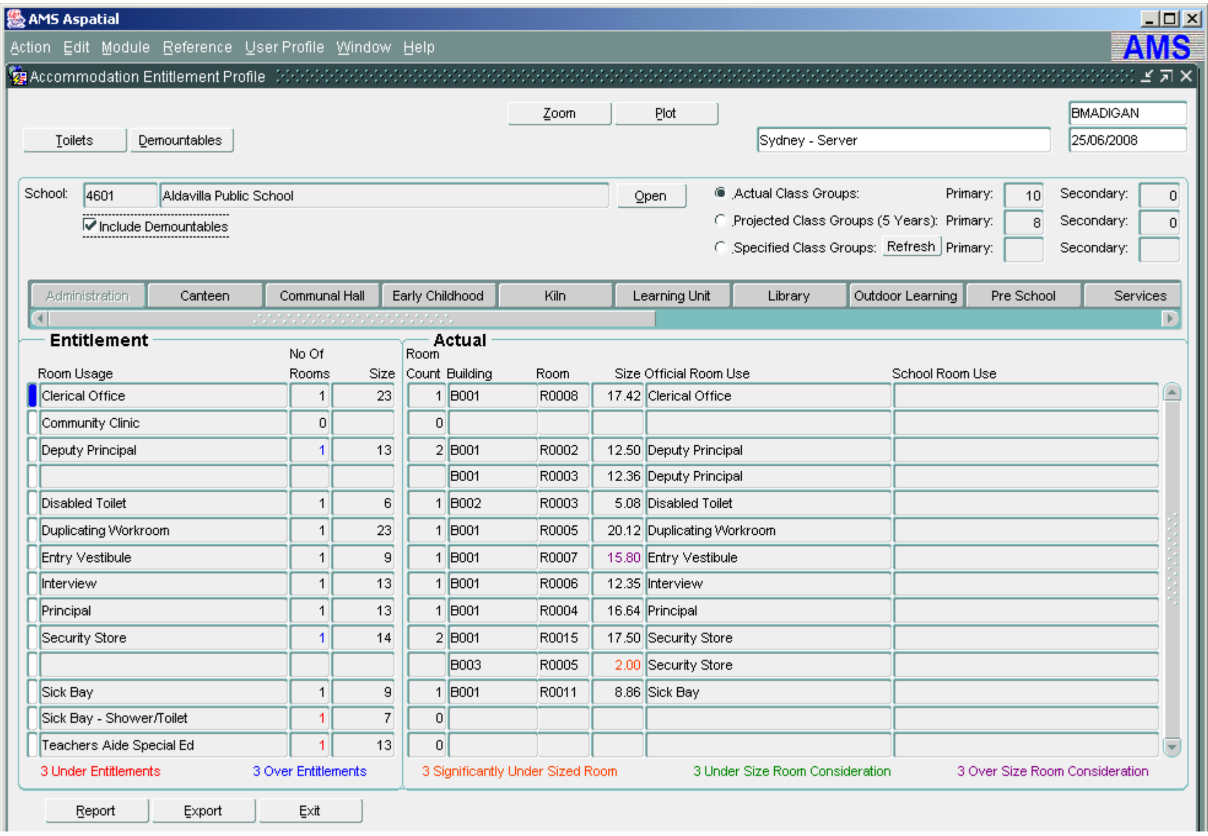

Figure 10: Comparison of entitlements to actual facilities. Such data assist with forward budgeting and program management activities.

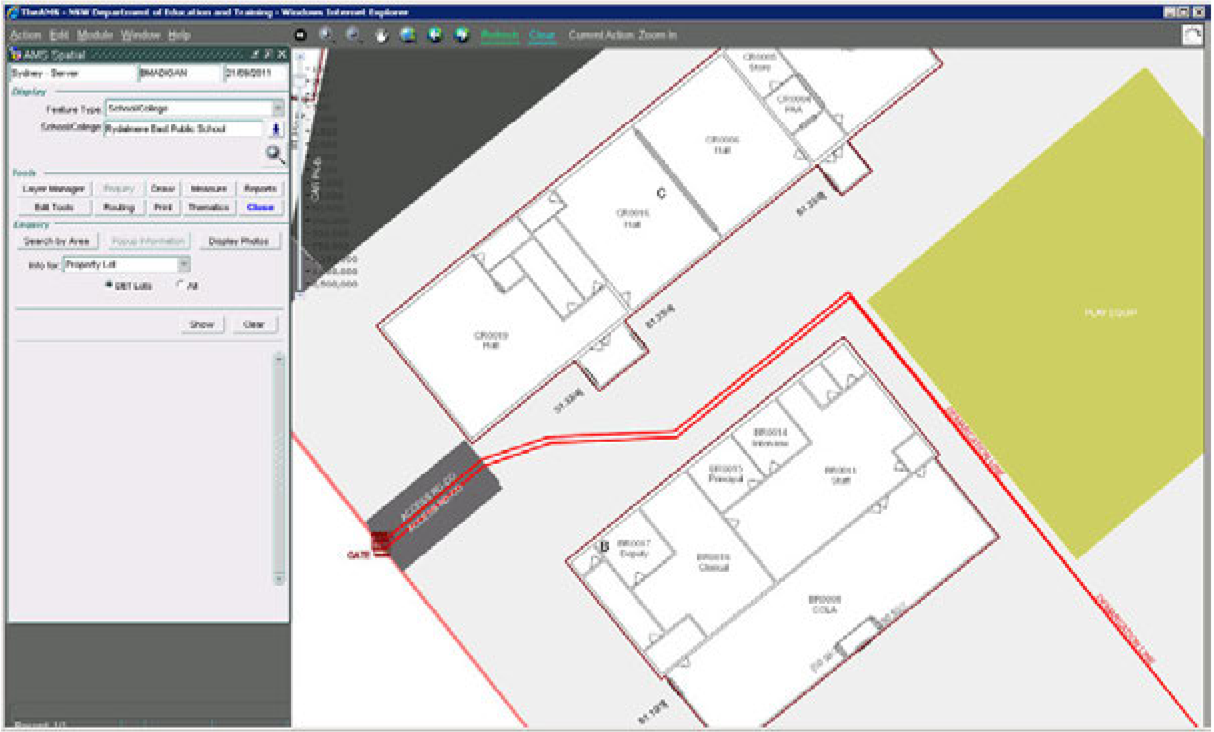

Figure 11: Detailed display of building facilities including access points and door swings – this level of spatial information is a highly valued input to Department activities including emergency evacuation, refurbishment and basic room usage.

Figure 12: Room usage thematic map.

5.3 Programs

The AMS program module provides a capability for the cradle-to-grave management of capital works projects using the Program Management Module (PMM). This module is at the

core of the current allocation of funding for all new school projects and was used extensively during the 'Building the Education Revolution' (BER). The PMM module caters for projects up to $3 million. During the BER, PMM managed in excess of 6,000 projects under the BER funding model with a future capacity to cater for projects expected to amount to a total spend of $1.4 billion.

An overview of the PMM module is as follows:

- Action list – 'my tasks for the day'.
- Nominations – typically provided by a school.
- Investigations – conducted by the DEC.
- Projects – the running of the Capital Works Project, i.e. DEC and Public Works staff.
- Tracking – 'status' of projects.
- Reporting – 'where are we up to'.

All of these activities exist within a 'process tree' with levels of security, access and authority. Digital signatures are used to track who has been responsible/involved within a project. Access and security is fully configurable and even caters for staff absence or higher duty appointments for periods of time.

Reporting is part of PMM from an internal DEC measurement perspective and includes such reporting levels as school, project, state and budget for activities such as:

- Nominations.
- Commitment/expenditure.
- Budgets and budget projections.
- Project milestones and progress including:
  - Milestone complete within requested time.
  - Milestone due with requested time.
  - Milestone completed late.
  - Milestone due and late.

Formats used are standard text formatted reports as well as bar graphs, pie charts and other display tools.

5.4 Finance

This module interfaces directly with DEC's SAP financial system as well as the maintenance, programs and demountable modules.

5.5 Spatial

This is a fully integrated set of information and functions that are available from within all AMS modules – an ability to always 'provide a picture'. Over 50 layers of spatial data are available within the system. Details down to every room in every building on every site are included. Demountables and site-based structures (e.g. gas tanks, bubblers and play equipment) are also located and shown within the system. The surveyed dimensions of every site have been compiled and recorded spatially in the GIS. All structures are located within each site.

5.6 Planning

This module provides support for the process of school and TAFE planning, including demographic analysis, catchment creation, enrolment data, entitlements and future projections (Figures 13-18). The planning module has a high spatial information component and allows users to spatially create areas of interest based on other spatial entities. The spatial relationship between schools in terms of access, distance and student profile are also key business inputs for school planners.

Included data originate from many sources including proposed re-zoning, transport and housing forecasts from the Department of Planning, development approvals and development control plans from councils, ‘live births’ from Medicare, industry and occupation projections from Access Economics and census data from the Australian Bureau of Statistics.

5.7 Maintenance

The maintenance module covers all maintenance information of all schools. Maintenance is undertaken by a panel of contractors who are responsible for all repairs, regular activity and reporting activity to DEC. The module records all maintenance activities undertaken by the panel. Each campus is inspected annually for condition assessment and the module also records this data as it is updated. It also provides reporting tools to allow for analysis of past, current (actual vs. estimate) and future (budgeting) purposes. The annual maintenance budget for DEC properties is currently in excess of $200 million.

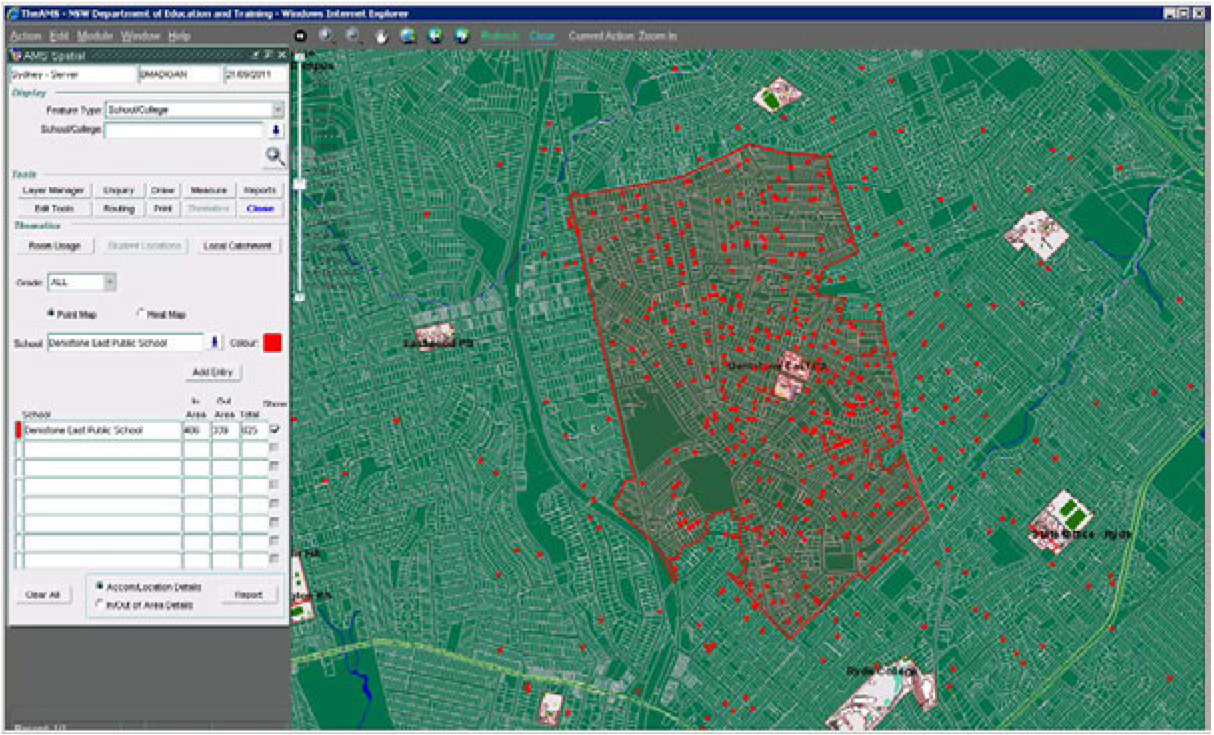

Figure 13: Location of student address details – used for ‘in’ and ‘out’ of catchment analysis within the planning module of AMS.

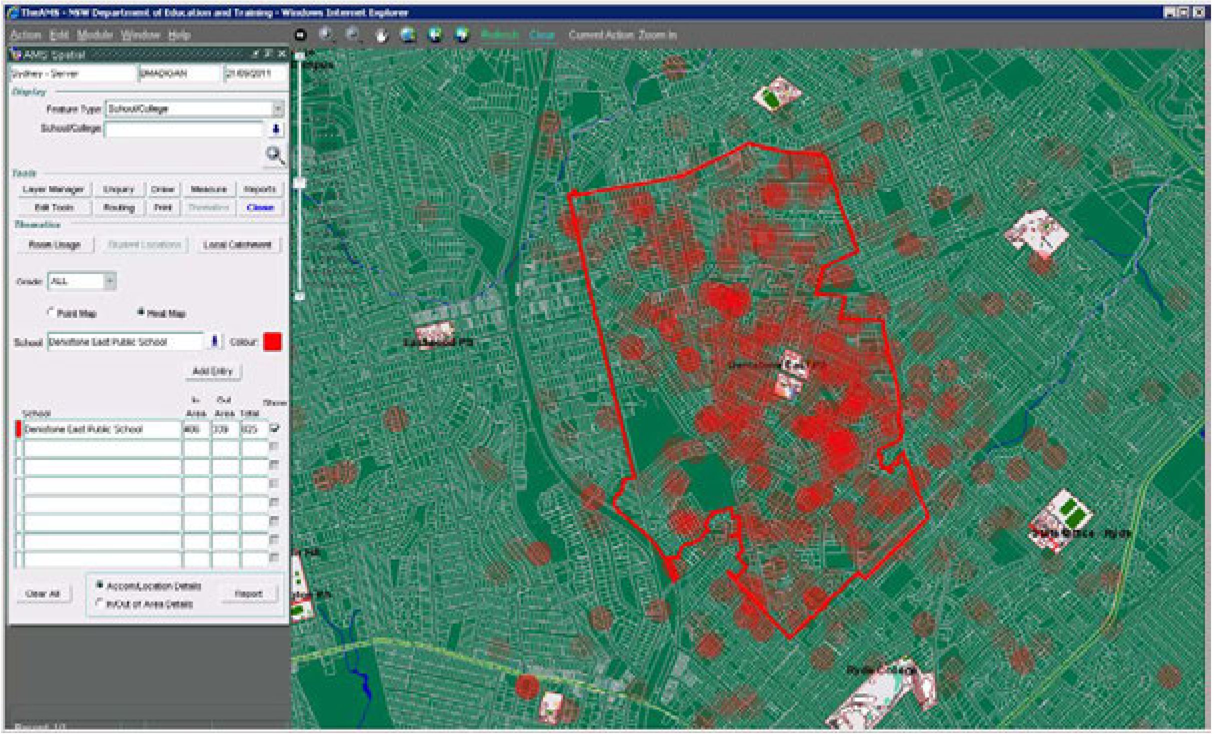

Figure 14: Facility for checking if a new student is located within an existing school catchment. This function also includes an interface to Google Maps.

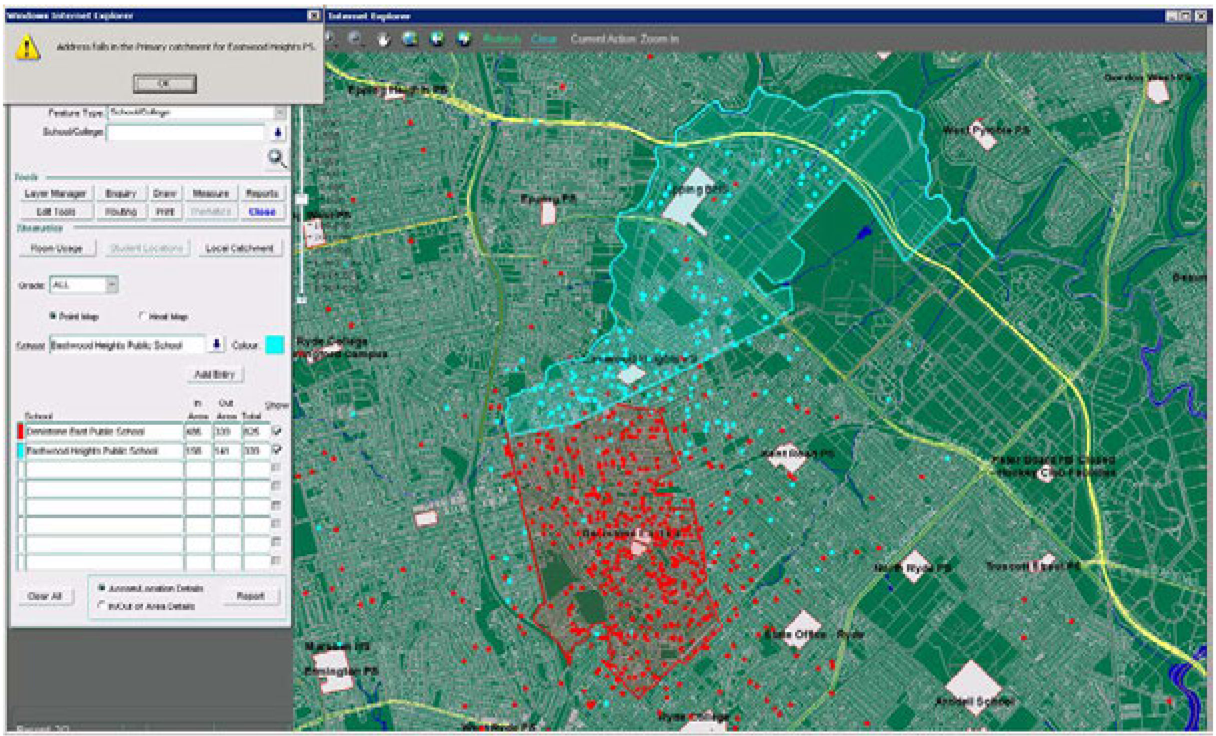

Figure 15: Linking of student address to a school catchment.

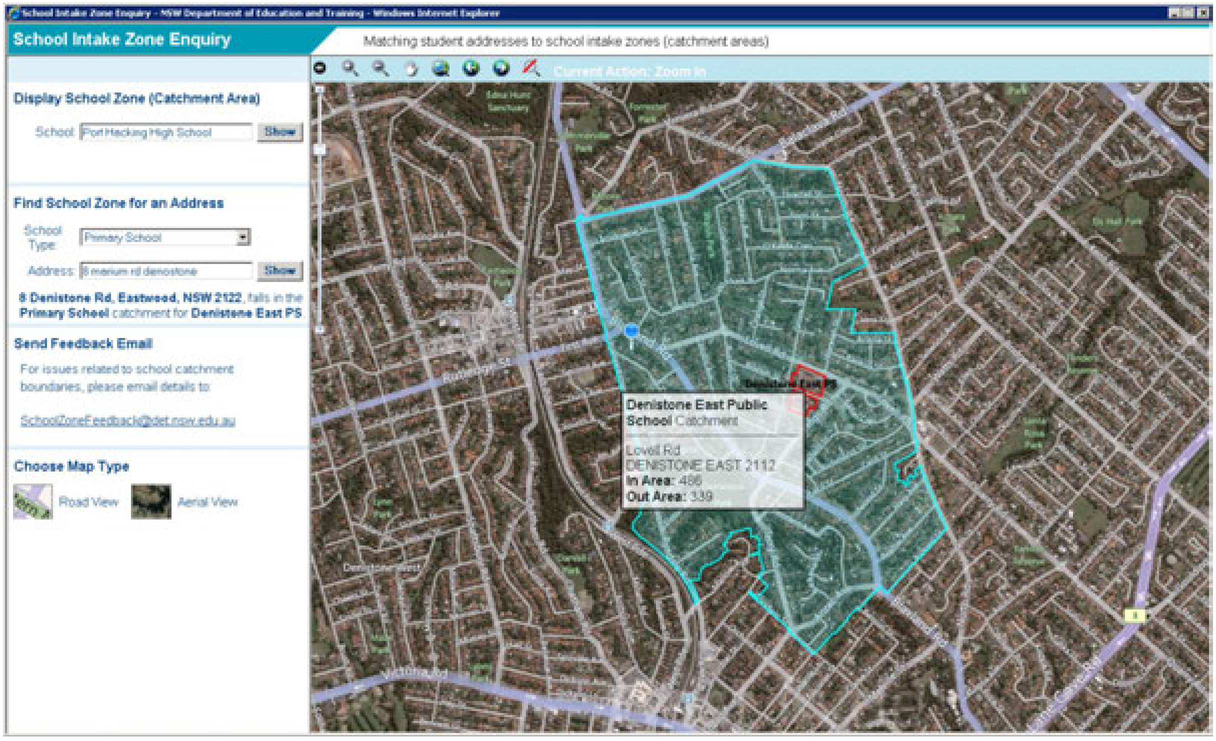

Figure 16: School intake zone enquiry – 'internet accessible' (integrated with Bing Maps).

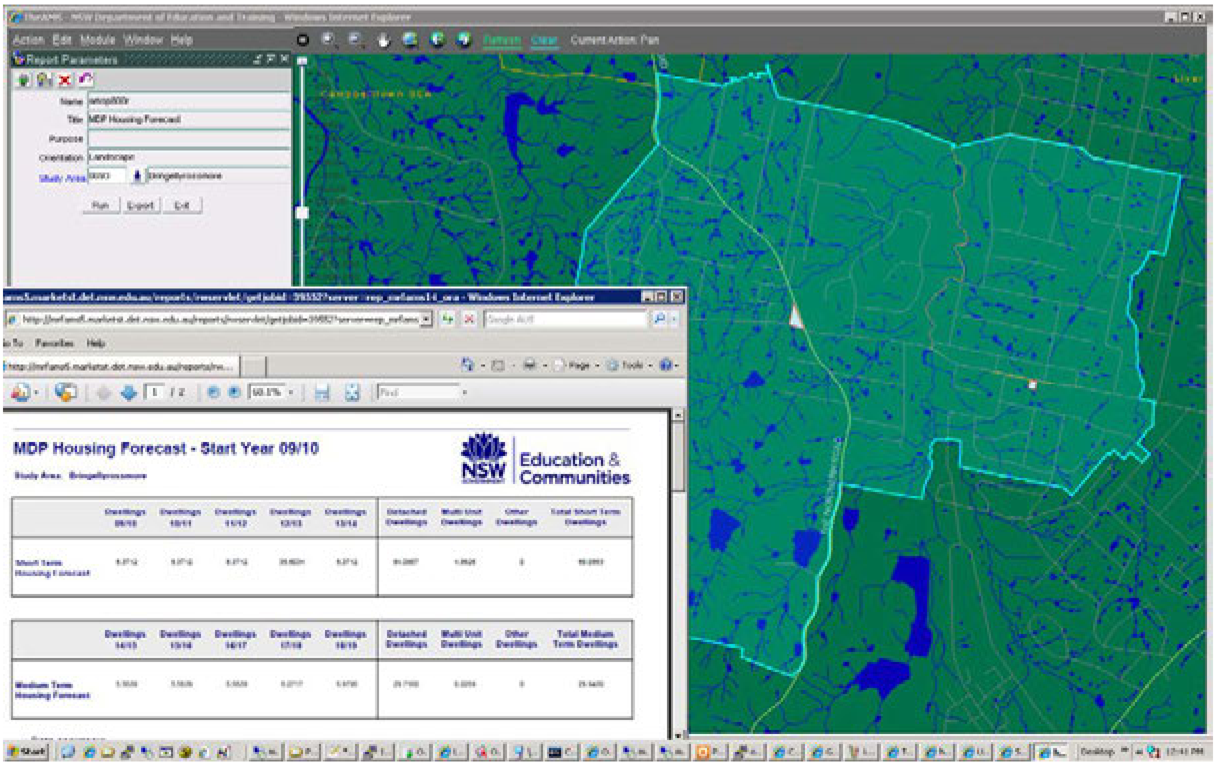

Figure 17: Metropolitan Development Plan (MDP) housing forecast for a study area.

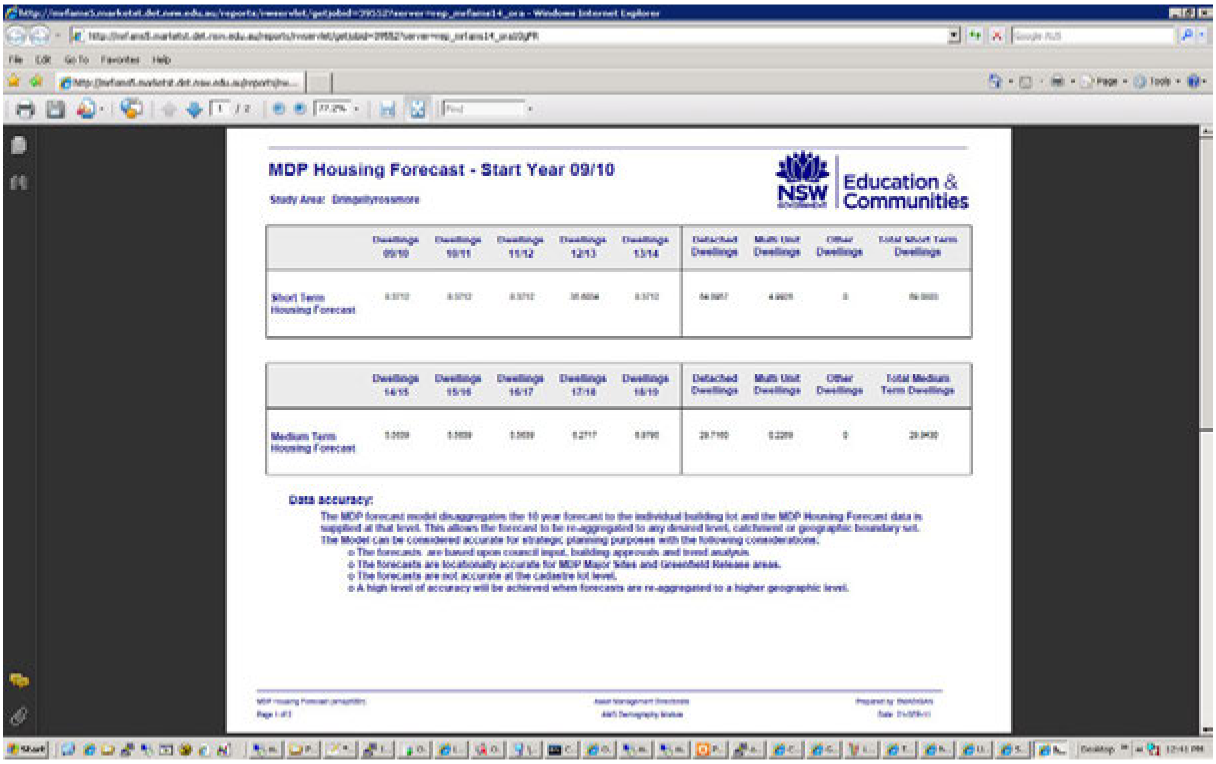

Figure 18: Metropolitan Development Plan (MDP) housing forecast for a study area detail.

5.8 Demountables

The demountables module provides support for both the management and operational tasks of demountable buildings. It provides for allocation of buildings (based on school needs and entitlements), production of work orders for site preparation, transportation and delivery of buildings including advice to crane operators, truck companies and maintenance contractors who are responsible for establishment (and re-establishment) of the site for the demountable. A tool is provided that even maps the ‘swing circle’ of the demountable while it is being lifted onto the site, incorporated with up-to-date aerial photos.

The spatial module allows Department staff to plan and locate demountable buildings on a site without the need for on-site inspections, resulting in further time savings and business efficiencies. This module is interfaced with the maintenance and planning modules for the purposes of maintenance. A spatial tool provides shortest-path analysis for input into transport costs and includes information on road hierarchy and bridge weights to ensure demountables are moved in the most efficient way between donor and recipient schools (Figures 19 & 20).

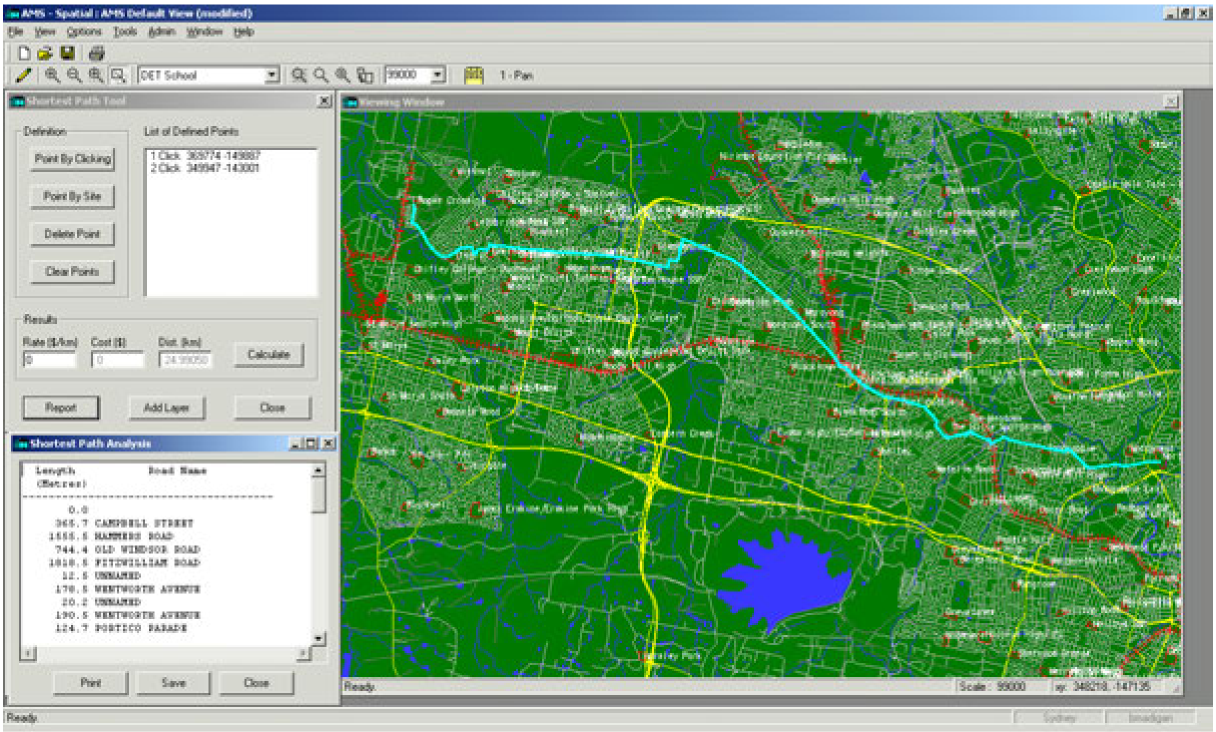

Figure 19: Shortest path between a 'donor' and 'recipient' school to assist with calculations of transport costs for demountable classrooms.

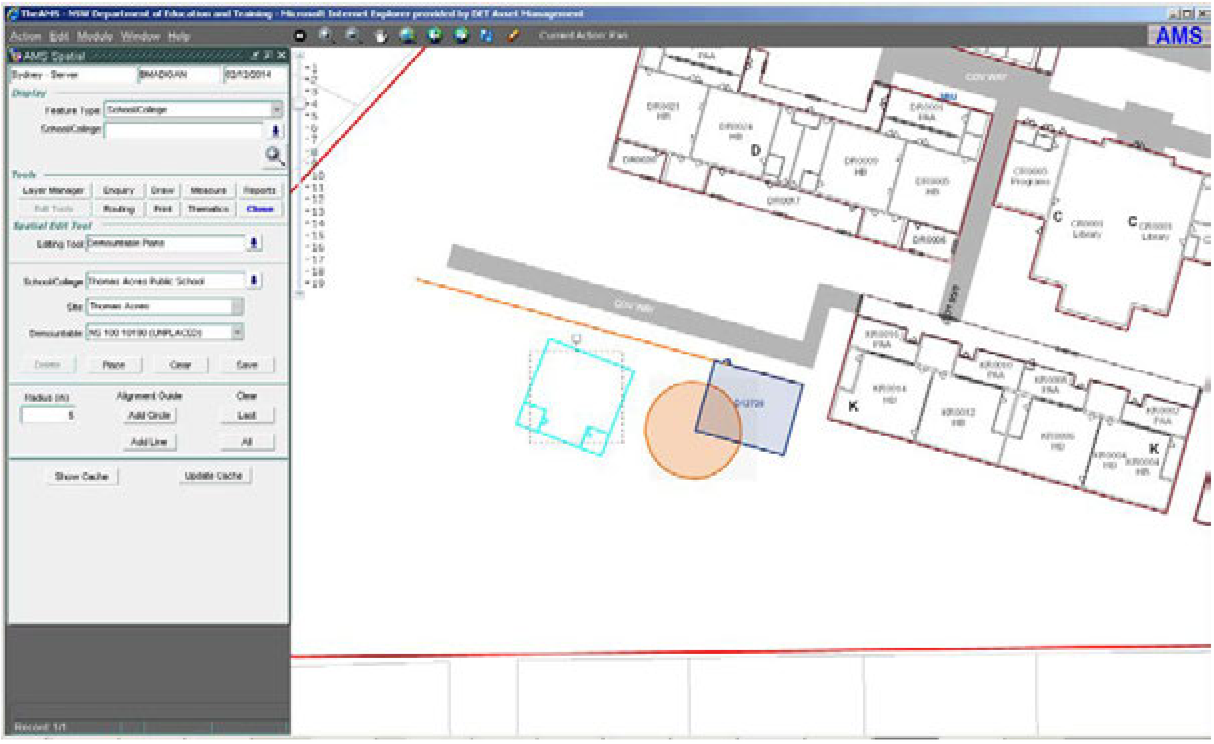

Figure 20: Example of the demountable placement tool using a predefined building module/template. Users can place the building, rotate and offset the building from existing structures and display the swing circle for the crane to allow for assessment of obstructions such as trees and overhead wires.

6 ASSETWHERE™

As a result of OMNILINK's experience with AMS, another client, Knox Grammar School at Wahroonga, advised that they needed a system to collate and display all their school property and building information. Their collection of plans covered a period of over 50 years and included survey, architectural and engineering drawings on various media and at various scales. Thus AssetWhere™ was born in 1999 as a Property Information Management System based on desktop GIS and a readily available relational database. Today OMNILINK has implemented the system in nearly 100 schools throughout Australia and New Zealand. This year has also seen its introduction into some British schools.

Version 4 of the software has just been released. It is browser-based and so can be viewed throughout campuses and on mobile devices. However, AssetWhere™ is not just for schools and educational establishments. OMNILINK has also installed the system in aged care facilities and golf courses, and is negotiating its adoption by commercial property interests. The system allows users to view all building and property information, internally and externally, in managed layers with navigation tools and the ability to point, click and extract information about the feature of interest. Layers include sub-layers of building/level/room, electrical, mechanical, telecommunication, fire, drainage, water reticulation, title information and landscaping. These layers can be viewed together in any combination (Figure 21).

Figure 21: AssetWhere™ Version 4, showing site features and layer manager.

As the user zooms in, further detail is displayed such as services and room details. Clicking within the room allows an interface displaying room attributes (Figure 22). It is worth noting the scale display in the lower right-hand corner and the scale bar in the lower left.

Figure 22: Room detail with attribute table.

The key to the utility of AssetWhere™ is the initial collation and sorting of information and understanding of the drawings. Clients are issued with an inventory of all relevant information after consultation with managers, staff, contractors and consultants. The system links to other software – fmXpert (a facilities maintenance software package), school records systems and financial systems. Consequently, users can view the location of an asset, retrieve its maintenance and financial history with a few clicks, issue a work order for repairs and, later, track the progress and performance of the repairs and the technician. Users can also view linked photographs and documents relating to assets (Figure 23).

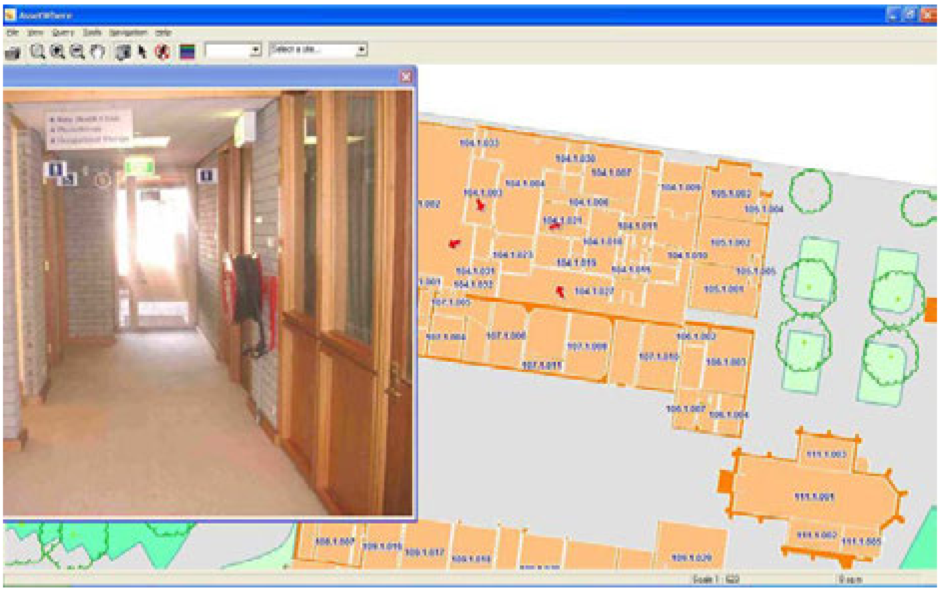

Figure 23: Linked photograph with room and service detail.

Property managers are able to use a measurement tool to retrieve distances and areas of rooms and designated sites. They are able to prepare a pdf plan of the location of asset features to scale to send to contractors for maintenance action that alleviates the need for the property manager to meet the contractor at the site thus saving valuable time. The system allows

managers to make decisions about priorities of work from a view of an incident. For example, consider liquid seeping from a site. Is it emanating from a break in a water pipe, a drainage pipe or a sewerage pipe? Without the system, this may have required excavation to determine. Experience has shown that AssetWhere™ repays itself in the first year of implementation and has a return on investment in subsequent years of over three times.

Further to a request from one of its New Zealand clients, OMNILINK developed a geospatial routing solution, which was named StudentWhere™ although it could just as easily be known as 'StakeholderWhere' for reasons to be described. The software initially located anonymised student addresses for the purposes of formulating school bus routes. Another client described a need to be able to identify lower socio-economic category students for the purposes of gaining government grants and this then became a marketing tool for schools to attract students from these socio-economic districts using census data. Subsequently this tool was used to assist businesses in finding appropriate locations for their market segments. Many firms use this type of software tool to assist them in deciding where to open new stores (e.g. McDonald's).

7 ASSET BASIS

The presented case studies highlight the importance of location to best-practice asset management. Service-oriented government departments generally require physical assets such as land and property to distribute and house these services. From a financial accounting point of view, departments are required to maintain records of these assets and their values for management and reporting. As responsible and leading employers, departments are required to maintain their physical property in safe working order. In addition, departments are caretakers of community assets and are required to keep them in a workable and possibly transferable condition.

As defined by accounting texts, an asset:

- Provides future economic benefits or services derived from its use or sale (i.e. it has value).
- Is controlled, i.e. an organisation has the capacity to gain benefit or to deny or provide benefit to others from the asset.
- Has resulted from some past transaction, internally or externally.

Thus this property information is in itself an asset for government. Given this, the asset requires protection and management, and it provides not only operational benefits but also a tool for risk mitigation. Table 1 illustrates how this is achieved.

Table 1: Risk-assurance matrix (© 2004 OMNILINK Pty Ltd – Garvin and Harkin, 2004).

Assurance	**Risk**
Improved process	Reinventing the wheel
Governance	Litigation
Shared learning	Knowledge hording (silos)
Customer retention	Customer defection
Competitive advantage	Loss of knowledge
Successful outcomes	Poor decisions
Increased efficiency	Inability to learn
Lower cost	Higher cost
Employee satisfaction	Lower productivity

8 THE DATA LIFECYCLE

Information is sourced from data. In any form, data has three broad dimensions: accuracy, currency, and efficacy. Accuracy and currency affect efficacy, but efficacy is an independent dimension that relates to fitness for purpose of the data requirements of the organisation. Accuracy relates to how close data is to the truth, and currency refers to the timeliness of the data (Garvin, 2007).

Accuracy and currency are affected by the data lifecycle (Figure 24) that involves people, processes and technology (Garvin and Harkin, 2004):

- Acquisition – the initial phase of collecting, capturing and collating the data by manual, digital or remote methods and whether it is done internally or externally, by instruction or purchased through independent suppliers.
- Integration – for textual data to interact there must be common fields in relational databases and with spatial data there also needs to be common map projections and parameters, common grid systems and origins and common scale factors to name a few. Then the data needs the ability to be brought together by the ability to access each other invisibly to the user from possibly different systems and stores.
- Maintenance – data that involves people and physical and man-made features inevitably change through natural forces, development and human intervention over different times. It therefore requires review for changes and revision, if necessary.
- Extraction – for data to become knowledge it must be revealed by querying and reporting from the appropriate data stores. This occurs programmatically and by the links and commonalities referred to above.
- Manipulation – publishing information from data requires it to be presented in a form with which users are comfortable. Reports, illustrations, graphs, charts and maps can be integrated, massaged and produced programmatically for user consumption.
- Archival – data that have become outdated or inaccurate are no longer efficacious and can be misleading and dangerous. It may have historic value and therefore needs to be retained so is retired from the current databases and held in different non-current stores for possible use in time series or other analysis. New data then replaces the old by the acquisition process.

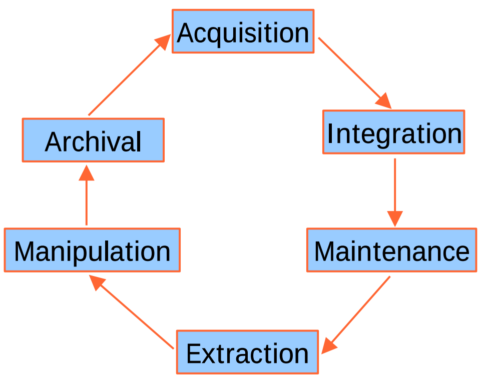

Figure 24: Data lifecycle.

9 THE VALUE OF ASSET DATA

Datasets alone have inherent financial value as an asset. Organisations are bought, sold and traded merely for the value of their data. While their datasets and information do not often

appear on their balance sheets as an identifiable item, they are included in their goodwill. Without data and information, how can organisations operate? Service providers particularly could not operate without information and data, e.g. customer and stakeholder lists, inventory and supplier lists, location details of work sites, routing and scheduling manifests, bills and accounts.

Asset data is a special subset of data. It can be more than an inventory of assets and a list of values for a balance sheet. It may contain data about the asset's condition that can assist with its written-down value, about the custodianship of the asset, about users and other stakeholders of the asset, its location and many other attributes. Its value in use includes operational, management and strategic factors. Furthermore, the value in use of data is limited if not negated if its currency is uncertain.

It is said that what cannot be measured, cannot be managed. Furthermore, one cannot manage what is not known, particularly if its location is unknown. Location is often overlooked, often assumed and is a critical factor in asset management. Location or spatial information and analysis are essential to effective asset, particularly property, management. Property and facilities management require current knowledge of where assets are located and how an asset relates to other assets and users, maintenance staff and management as well as its condition, its history, its likely failure and replacement date and its construction type, just to name a few attributes.

There is anecdotal evidence of the value of the asset management systems implemented in the Department of Education and Early Childhood Development, Victoria, SAMS and the NSW AMS. In the case of SAMS, the cost of implementation that included data capture and validation, system development, systems and data integration and final roll-out amounting to several million dollars was repaid by savings in cleaning contracts in the first year of SAMS operation and accumulated thereafter. A similar cost and benefit was experienced in NSW by cleaning savings as well as savings in demountable classroom movements and campus planning. Less enumerated benefits for both systems include the retention of corporate knowledge, risk mitigation and the ability to link asset management to other systems such as financials.

10 ONGOING ASSET DATA MAINTENANCE

Buildings and infrastructure have a design and material lifetime and are removed, refurbished and replaced. Rooms are re-organised, and service infrastructure replaced and re-routed. Standards for construction change and require additions and renovations. Normal wear and tear dictates changes. So the asset information should change with assets to maintain currency, otherwise risk occurs. Financial risk stems from extra time and charges from contractors unable to locate the asset. It could also arise from injury and compensation caused by inadvertently connecting with the wrong infrastructure resulting in explosion, electrocution or exposure to raw sewage, thereby also causing legal and reputational risk. While the likelihood of this could be assumed to be low, the consequences could be catastrophic, especially in a school environment in which the school has a distinct duty of care.

The data maintenance process requires collection and collation of data for assets that have materially changed. Material change is defined by the user but includes major changes such as new buildings, additions to and demolition of buildings and site service infrastructure

construction and minor changes including removal and addition of walls, changes to room openings and small service adjustments. The source of this data also needs to be decided on when specification for data capture is initialised. Once received by the agency responsible for storage of the information, the data is required to be validated for consistency and then integrated into the appropriate storage receptacles such as a database and/or a GIS.

Data maintenance should be undertaken by trained technical resources that include database administrators, Computer-Aided Drafting (CAD) operators and GIS analysts. Data capture for maintenance can be by on-site measurement using qualified surveying professionals or school staff or from as-built plans or sketches. There is a balance between the data capture process and the integration process. For qualified field capture as opposed to design data capture, less office procedures are required as the data has been formatted and is relatively straightforward to integrate into a system. Alternatively, capture from plans or sketches requires more intensive processing to integrate but is less expensive to capture.

10.1 Maintenance Costs

Indicative data maintenance costs for NSW include major works data updates that are required to be provided by the building contractor who must engage a surveyor from a departmental panel, adding approximately $100,000 per annum to construction costs. The annual costs for data integration are approximately $200,000 plus about $800,000 for extra site surveying and CAD processing. About 300 sites are updated annually from major and minor works out of a total of 3,500 sites that include TAFE colleges as well as schools. Additionally, about another 300 sites are updated annually by maintenance contractors.

As previously mentioned, savings from asset systems in other states have amounted to millions of dollars annually from cleaning contracts alone. Add to this efficiency savings in management of portable classroom movements, school planning, property maintenance and refurbishment programs, and property administration generally. In particular, in NSW the Department has indicated that, because of the manner in which data is stored and kept current, building and site infrastructure valuation has become easier and more precise, providing a more accurate figure with less site visits and time (therefore less cost) to calculate the total. The total annual savings for valuations in NSW have been estimated to be about $4 million.

Management of portable classroom movements is served by AMS resulting in significant savings. These ‘demountables’ each have a unique identifier and their location is registered against the site they currently occupy. As they are removed from a site, this data is updated, thus facilitating their management and placement in a new recipient school. Additionally, road routes between the donor and recipient sites are optimised using road centreline data that has bridge and road capacity limits stored, saving embarrassing and costly accidents and delays. AMS has eliminated a significantly manual process that was prone to entry errors and eliminated uncertainty to transport routing that cost the Department time and funds. According to an advisability study on the Asset Management System in October 1994, the Department of Education estimates savings to be $433,500 per annum (adjusted for CPI by 70%).

School planning has also been simplified in that AMS provides a platform that displays land zones (new residential release areas in particular), housing and subdivision development approvals, population growth statistics by ABS census geographical cells and birth data on a topographical underlay to assist with location of new schools. School planners use this

integrated data to provide information to select suitable sites for new schools whereas prior to AMS, hardcopy plans and disparate data from various sources required time-consuming effort to reach their conclusions with the possibility that their choice was not as good as that which could be reached with AMS. According to the same advisability study on the Asset Management System in October 1994, the Department of Education estimates savings to be $110,500 per annum (adjusted for CPI by 70%).

School property maintenance and refurbishment programs are also supported by AMS. This is a major budgetary item of over $200 million per annum direct expenditure and is totally reliant on current data from the system and that reported to AMS by contractors. The current state of school buildings and infrastructure affects their valuation so this reporting activity (data maintenance) is essential to maintaining an accurate database that allows a realistic value of school property, which is currently approximately $28 billion in NSW. This valuation in turn allows for evidence-based decision making regarding refurbishment, rebuilding or disposal. A summary of estimated savings for various activities resulting from AMS (based on the advisability study mentioned above) is shown in Table 2.

Table 2: AMS savings estimates in dollars (Clarke, 1994).

Activity	1994 savings estimates	CPI adjusted 2014 estimates
Asset management	5,000,000	8,500,000
Building code compliance	1,100,000	1,870,000
Maintenance	1,000,000	1,700,000
Demountable transport & siting	255,000	434,000
Administration productivity	227,000	386,000
Planning, review & queries	65,000	110,000
Transport claims	39,000	66,000
Ministerial briefings	26,000	44,000
Improved data reporting	22,000	37,000
Systems operating costs	17,000	29,000
Valuations	–	4,000,000
Total		**17,176,000**

There are also numerous intangible or unquantifiable benefits and savings such as estimates for replacement of destroyed or reinstatement of vandalised buildings, better identification of workplace health and safety issues, ready calculation of capital works nominations, better definition of school bus services and ease of review of school catchment boundaries. Additionally, there are extra benefits that emanate from ease of access to the asset information, the ability to distribute the information, the information being available to the public and the facilitated planning from these strengths that allow time savings.

10.2 Benefit vs. Cost

As indicated previously, unless asset data maintenance is ongoing for currency of the asset data, there will be a resultant lack of confidence in the system and concomitant risks that out-of-date data in the asset system will cause damage to life and property so there are significant savings from lower opportunity costs. It can also be argued that savings from data maintenance can account for much of the total savings and are possibly equal to the savings achieved from the system in total. However, for this analysis, it is assumed that the benefits from data maintenance are proportional to the number of sites that are altered annually in relation to the total number of sites. That is, 600 sites relative to 3,500 sites or 17%. For a total NSW system benefit of $17.2 million, some of the data maintenance benefit is assumed to be a minimum of about $2.95 million. For NSW, the total cost of data maintenance for

AMS is a maximum of about $1.1 million. So the minimum benefit/cost ratio from NSW asset data maintenance is approximately 2.7 to 1, i.e. a return on investment of about 270%.

11 CONCLUDING REMARKS

The AMS project has evolved over 20 years from originally being focussed on strategic management requirements to now being a key, integral part of many operational and tactical systems within the Properties Directorate of the Department of Education and Communities. Many business functions of the Department are now solely conducted using the AMS system. It is a comprehensive system that facilitates the planning, project management, maintenance, financial and operational administration of every school (from one teacher schools to large multi-campus high schools) and TAFE campuses in NSW.

Since its inception the benefits realised by the Department have exceeded their original expectations through the delivery of operational efficiencies, the adoption of new asset management strategies and improved information flows. The project motto has always been "User Driven – Technology Enabled" and this approach has always resulted in an outcome of excellence that meets user and business requirements and enlarged expectations.

The system has been developed with a scalable and extensible architecture and database model that makes the addition of new data an extremely simple process. At no point during the life of the AMS project has there been a need to redesign that database model – a testament to the design of the Department and the original development team.

Geospatial enablement gives AMS extra functionality and usability. Rooms and other spaces, buildings, sites and some plant and equipment details can be readily located in relation to each other as well as to the communities these assets serve. Plans and drawings are sidelined no longer to the filing cabinet but act as essential components of the system and provide users with a real-world view of financial and management aspects of the Department's business. Surveyors play an essential role in the whole process from initial data capture and positioning to ongoing data maintenance. Their professional skills in measurement and observation as well as their experience with buildings and drawings are essential to providing accurate attribute, dimension and locational data.

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Working with Mobile Laser Scanning Providers to Optimise Value

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ABSTRACT

Mobile Laser Scanning (MLS) is a technology that has rapidly developed since its introduction to Australia in 2009. The technology provides the surveying industry enormous benefits in terms of safety, speed of data capture and richness of collected datasets. In the last few years, MLS technology has proven its capability with a large range of road and rail corridor surveys that have provided engineering-accurate surveying results. There is a growing interest in the market to understand how traditional survey companies can best take advantage of MLS technology and how they can work with MLS providers to achieve cost-effective solutions for corridor surveys. This paper addresses the issue of working with MLS data suppliers by discussing the capabilities of MLS technology (what accuracy can be achieved and what deliverables are available), the requirements of a successful MLS survey (what is required to ensure the MLS survey meets its goals), the limitations of the technology (so that they can plan to capture data using alternative approaches) and the role of data processing, data extraction and survey deliverables. The aim of the paper is to provide surveyors with some basic information about MLS surveys in order to understand the survey principles, processes and deliverables available. Understanding this information will help surveyors assess whether MLS should be considered for any corridor survey task, enable them to discuss the job requirements with their MLS supplier and better understand the role they can play in any future MLS survey.

KEYWORDS: *Mobile Laser Scanning, MLS, MLS control surveys, ground feature models, MLS data extraction*.

1 INTRODUCTION

Mobile Laser Scanning (MLS) technology emerged in the Australian survey market in 2009 and has proven to be an ideal tool for high-accuracy, corridor surveys that provide a 'data-rich' deliverable ideal for data mining. MLS is a Light Detection and Ranging (LiDAR) technology combining the principles to airborne LiDAR with the accuracies closer to Terrestrial Laser Scanning (TLS). The benefits that MLS offers include the increased safety for road/rail workers, more detailed and comprehensive measurement of all features on the corridor, high speed of data acquisition and the accuracy of the final result.

Since 2010, MLS has been used by most Roads Departments around Australia. Corridor surveys vary from rural and suburban roads to highways and freeways. They have also been

used for railway surveys, and for other linear features such as powerline corridors and canals. The MLS surveys have been independently validated and have proven to achieve engineering accuracies.

From a technology point of view, these are still early days. A range of different systems are available in small numbers around the world. Some have been designed for ‘asset capture’ surveys, while others are developed to provide ‘survey-accurate’ MLS data. Many of these systems have been developed by the scanner suppliers, some adapted from airborne LiDAR equipment, and others by third party vendors who have assembled systems from ‘of-the-shelf’ components (i.e. scanners, IMUs, GNSS and cameras). Major surveying suppliers and software providers are working hard to develop ‘user friendly’ MLS systems for the wider market.

Over the last few years a number of MLS service providers have emerged on the market. Each of them has made a major investment in technology, software and hardware development and training of personnel. They are living in a world of steep learning curves, and constantly strive to optimise the output of their systems in a cost-effective manner.

This business model is not appealing to many traditional survey companies. However, many of them are involved in corridor survey work and are looking to the MLS suppliers in the same way as they look to suppliers of aerial photogrammetry or hydrographic data. There is a growing awareness of MLS in the market, and a desire to understand what data a MLS supplier can provide and the role that they can play in facilitating and supporting MLS surveys.

This paper aims to address these issues by:

- Providing an overview of the steps within a MLS survey.
- Demonstrating the dynamic nature of the technology by briefly describing some recent developments that have occurred to meet customers’ needs.
- Discussing the strengths of MLS surveys, and where they provide outstanding survey results.
- Explaining some of the limitations of MLS technology, and the role that traditional surveys play to address these limitations.
- Providing an overview of the functions that traditional surveys play to support and facilitate a MLS survey, and how survey companies can become involved in the MLS survey process and data deliverables.

2 MLS SURVEY OVERVIEW

Mobile Laser Scanning is another survey tool in the surveyor’s arsenal that needs to be treated with the same principles that apply to any other survey measurement. Surveys needs to be connected to local control and independently verified. For high-accuracy MLS surveys, the survey vehicle is positioned with a dual-frequency GNSS receiver (1 Hz), augmented with an Inertial Measurement Unit (IMU) that provides positions and attitude at 200 Hz. Data is processed in the WGS84 datum and transformed into the local reference frame, by measuring some local targeted control points.

There are a range of error sources that affect the system. Many of them can be minimised by correct calibration of the equipment. Some major error sources that cannot be corrected

through a calibration process include the GNSS positioning errors caused by multipath, changes in satellite configuration and heighting errors from incorrect geoid modelling. There are a couple of methods currently used to monitor and correct these errors. One commonly used method is the multi-target approach, which requires multiple targets to be placed along the road corridor to monitor the difference between GNSS-derived positions and target positions. An alternative approach is 'Multi-Pass' which uses multiple passes along the same corridor (under different satellite conditions) to monitor the deviation and magnitude of the GNSS trajectory between known target points. These methods have been described in Eckels and Nolan (2013).

Table 1 shows the basic steps involved in a MLS survey.

Table 1: The basic steps of a MLS survey.

	Action	Description
1	Check control – Horizontal and vertical	As the MLS survey needs to be connected to the local control, it is important to verify that local marks exist, that they have not been disturbed, and whether or not they can be used for GNSS base stations. Checking of the marks is carried out by GNSS surveys and traversing. Digital levelling is carried out where required.
2	Target placement	Targets are placed on the road shoulder, where they can be identified in the scan. For some surveys targets are required every 400 m. For other approaches, targets need to be placed each 5 km. It is important to note that placement of targets and validation points is one of the most labour-intensive and time-consuming processes of any MLS survey, and often requires applications of road occupancy licenses and traffic management.
3	Provide validation points	Many surveys require independent QQ string or validation points along the corridor. These points are used to take independent survey measurements across the road profile to check the cross section of the scanned model.
4	Scan and collect data – GNSS base stations	During the scanning process, a couple of GNSS base stations are established on local control. This enables high-accuracy phase differential GNSS positioning of the working vehicle. Scanning takes place from the scanning vehicle and all data is logged in an on-board computer.
5	Process the point cloud – including pinning to control	Collected data (can be many 100 Gb) is downloaded, and a point cloud is processed. The data is transformed into the local reference frame using the target information provided.
6	Extracting data from the point cloud	One of the most labour-intensive components of the survey is the extraction of data from the point cloud. This process highlights the differences between LiDAR surveys (which have lots of points without attributes) and standard surveys (which have few points with associated attributes).
7	Delivering the data in the correct format for customer	A key issue for all customers is to deliver the extracted data in their documented CAD format. This process is not always straightforward and takes experience and knowledge to ensure the final deliverable is acceptable.

3 FAST DEVELOPING TECHNOLOGY

The procedures used for MLS surveys are constantly changing and improving. As consumers and end-users review the MLS process, they seek to improve and automate as many procedures as possible. One example of these changes can be found in the approach to target design over the last four years. The first major improvement was to increase the size of the targets (0.6 m x 0.6 m) to accommodate the collection of scan data at any speed. Recent developments have enabled Terrestrial Laser Scanning to be used to create a 'target surface'

on the road. These can be a good alternative, as they can be measured from outside the road corridor, without having to block a lane.

Significant amounts of research and development are being conducted to address the labour-intensive and cost-inefficient components of current MLS survey methodology. The two most time consuming components of common MLS methodology are (a) setting out the targets for monitoring the MLS point cloud (methods such as 'Multi-Pass' have been developed by McMullen Nolan Group (MNG) to address this issue – see Eckels and Nolan, 2013), and (b) extraction of point and line CAD data from the point cloud itself.

Many software companies are working hard to automate the extraction process, but this process will take some years to perfect. Our experience has shown that software can currently automate 80% of an extraction process, however, the time saved (and often more) is spent in manually sifting through the data to find where the 20% of errors actually are. Much of the extraction therefore is still a manual process.

In addition to optimising current processes to meet survey requirements, a range of applications are emerging for project visualisation. Coloured point clouds (Figure 1) are now available. As these point clouds have such a dense richness of data, they are being used as 3D imagery – offering 'photo-like representations' of the existing survey corridor. However, these are not photographs, but a dense cloud of measurable points. This process has been even further advanced with Euclideon's release of 'Solidscan', which manipulates the point cloud to appear exactly like a photograph. These sorts of technologies will continue to explore the 'visualisation' aspect of point clouds for planners, architects and project managers.

Another exciting development is the combination of all types of LiDAR data for a total corridor survey. Recently MNG has combined MLS corridor surveys with airborne LiDAR along road corridors in WA. MLS provides high-accuracy road surface and ground feature modelling information, while airborne LiDAR covers a wider swath of data over the entire corridor. The aerial data provides enhanced vertical measurements for Digital Terrain Model (DTM) determination and offers measurements of some features that have been obstructed from the road (e.g. behind noise walls). In the next five years, we should expect MLS survey processes and data extraction methodologies to improve, to become even more automated, productive and cost-effective.

Figure 1: Colour point cloud from Sorrento, WA.

4 SURVEYS SUITED TO MLS

MLS can be used for a range of surveys. For some types of surveys MLS offers exceptional value as it provides data that is extremely difficult to collect using traditional surveying technology. Some examples of these types of surveys are outlined in this section.

4.1 Road Surface – Kerb to Kerb

MLS is ideal for collecting high-density, high-accuracy data on any road surface kerb to kerb. This data can be collected without a surveyor having to work on the street. It does not require traffic management, night work or closed lanes. The accuracy of the survey and point density is high, as the distance from the scanner to the road surface is minimised.

Outputs from such surveys include the DTM of the road surface, e.g. to identify road rutting, surface camber, subsidence and slumping (Figure 2), and road utilities such as road furniture and drainage pits in order to determine status, dimensions, location and condition of assets (Figure 3). The only impediments to these surveys are obstructions on the roads caused by other traffic. However, these issues can be overcome by adopting techniques such as ‘Multi-Pass’.

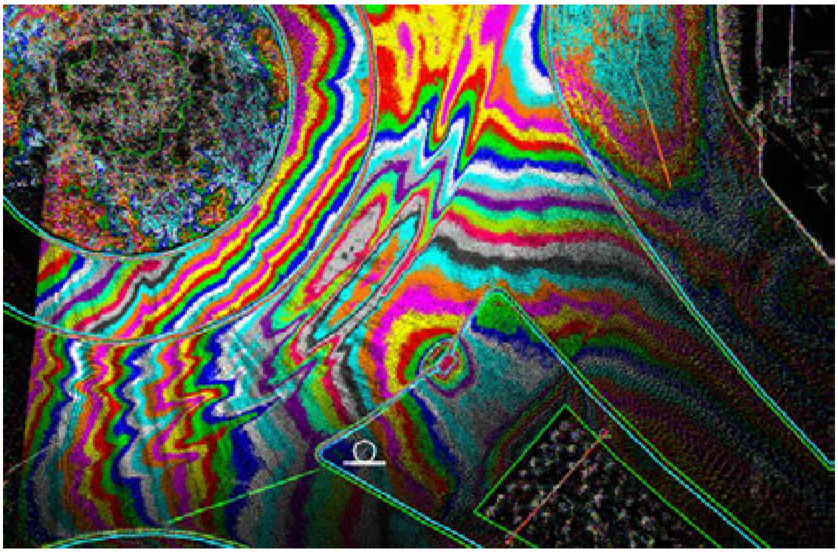

Figure 2: 1 cm contour of a roundabout.

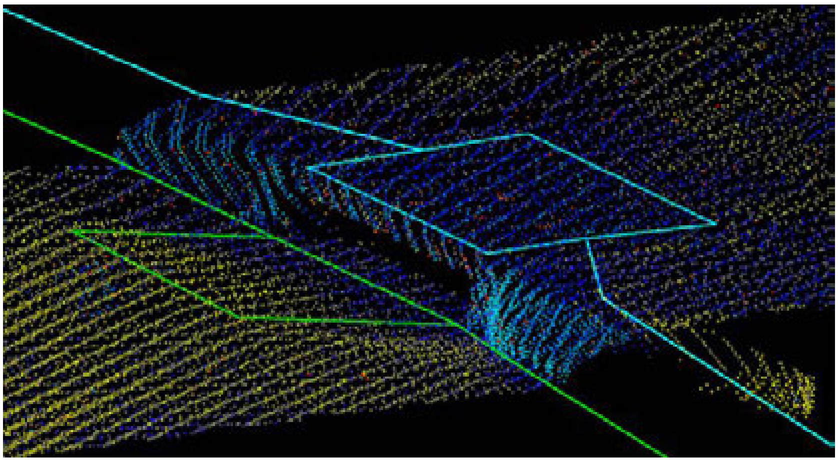

Figure 3: MLS measures a gully pit.

4.2 Hard-to-Access Structures

MLS surveys can accurately measure areas of difficult access quickly and productively. They can be used to accurately measure thousands of assets such as power poles, street signs and the clearance for overhead bridges at driving speed (Figure 4). They provide a comprehensive picture of assets that are difficult to reach with any other traditional survey technology.

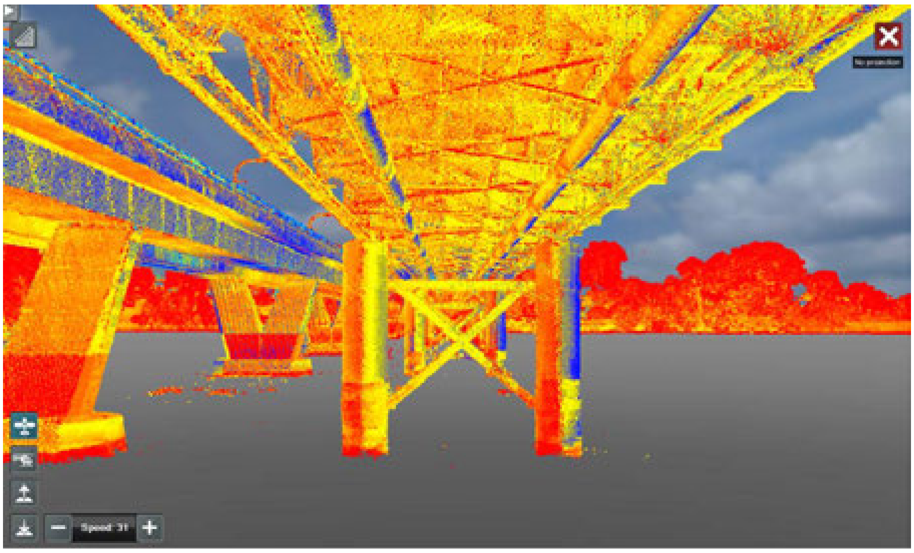

Figure 4: Measurement under the deck of Shoalhaven Bridge.

4.3 Sensitive Areas

MLS has also been employed in ‘sensitive’ areas, where high-accuracy survey work is required without alerting the neighbourhood to the presence of surveyors. A significant amount of survey data can be collected quickly and accurately just by driving past. As MLS surveys are limited to ‘line of sight’, features obstructed by fences and walls cannot be measured with MLS.

4.4 Clash Detection Surveys

MLS is an ideal tool for ‘clash detection’ surveys along a road or rail corridor (Figure 5). In a track maintenance program, the tracks are monitored to ensure that they have not moved from their design alignment. The vertical profile of the track can be significantly changed over the years as new ballast is placed under the tracks. When new rolling stock is introduced to the rail system, track managers are always concerned that clashes between fixed structures on the current track alignment and the rolling stock may occur.

On the road network, overhead clearance surveys are constantly undertaken. Clearances change as new layers of asphalt are applied to the road surface, and as the structures themselves settle. The clearance information is important to help route ‘oversize’ vehicles from one location to another, without risking a collision with the structure or damaging the load on the truck.

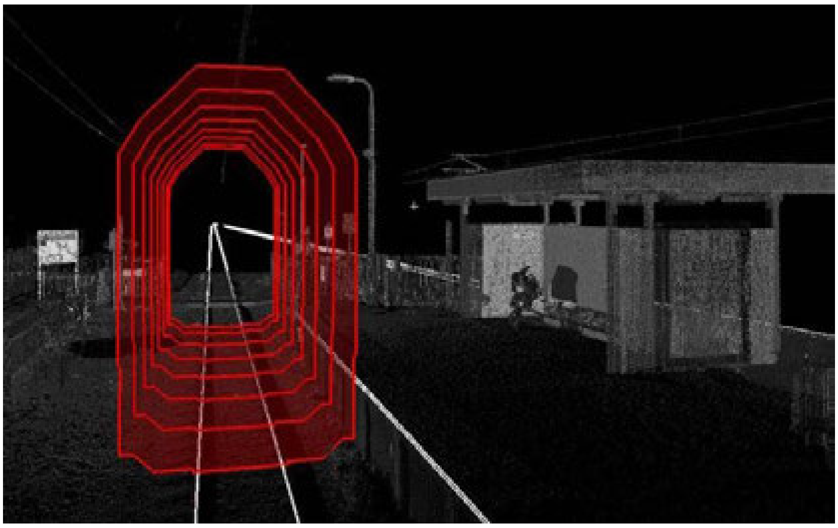

Figure 5: Clash detection survey.

5 PROJECTS REQUIRING BOTH MLS AND TRADITIONAL SURVEY

The majority of corridor surveys require a combination of both MLS and traditional survey methodology. As mentioned in section 2, traditional surveys are required to check the survey control and place targets and independent model validation control points (for QQ strings). However, many corridor surveys require survey information that is difficult to collect with MLS.

Ground features that are difficult to measure with MLS include:

- Utility manholes and access points. These can be hard to identify in the scan, especially if they are surrounded by grass, or partially covered by dirt. In many cases, our experience tells us that they are more easily identified by field personnel on the ground, and picked up using Real Time Kinematic (RTK) GNSS or other survey techniques.
- Drainage infrastructure, including pipes and culverts (Figure 6). Although MLS can identify the location, size and orientation of gully pits and headwalls, it is impossible to measure the pipe inverts or the direction of drainage. All of this work needs to be completed through a field survey.
- Natural surfaces in areas of thick vegetation. In areas of light vegetation, some parts of the laser signal will penetrate to ground level. By selecting the ‘lowest’ points on the natural surface, an accurate DTM can be generated. In areas of thick and dense vegetation or grassland, however, it may be impossible to measure any natural surface points using MLS. These areas can be identified and a field survey required measuring the appropriate number of ground shots in order to determine the shape of the topography. Often Airborne Laser Scanning (ALS), where points are measured down through the grass or canopy from the air, is more effective in determining the ground surface.
- A field survey is required in any areas that are obstructed from the scanner (Figure 7). These include areas behind noise walls, areas above embankments, areas obstructed by jersey kerbs or large billboard signs. In some inner city areas, these may also include features that are hidden behind parked cars.
- Features that need accurate positioning and are located more than 20-30 m from the scan vehicle. The errors in positioning gathered from scanning increase with distance from the scanner itself. The absolute position of any feature within should be able to be determined

to approximately ±15 mm accuracy. Once features lie more than 30 m from the scanner, their absolute accuracy will start to decrease. At about 80 m range, we would only expect positioning accuracy at the decimetre level. If the corridor survey requires features that may be located more than 30 m from the road (e.g. fence lines or utilities), a field survey will be required to collect them accurately.

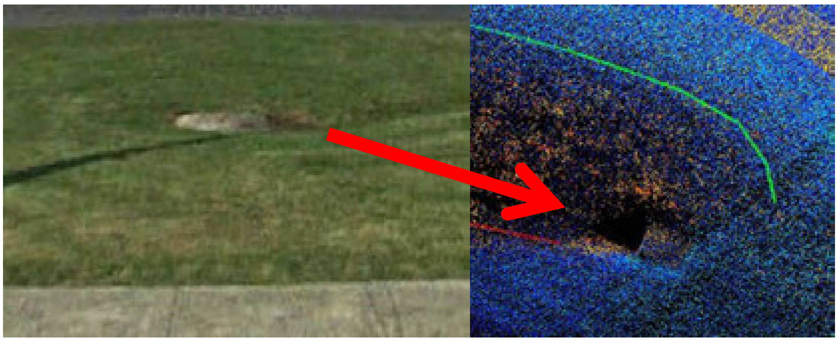

Figure 6: Drainage pipe in imagery and laser scan.

Figure 7: Pit location behind a bus shelter – requires field survey.

As can be seen from these examples, traditional surveying plays an important role in supporting and supplementing the MLS data for corridor surveys.

6 PARTICIPATING IN A MLS SURVEY

Working with MLS suppliers can be of benefit to both parties. The MLS supplier can concentrate their efforts on the MLS survey, without having to ship equipment and survey crews around the country to carry out standard survey procedures. Local surveying companies and organisations can provide high-accuracy corridor surveys to their clients using MLS data. They can become involved in the MLS survey at many levels to ensure the survey is completed to the client’s required accuracy. Local knowledge of the area and the survey marks can greatly assist the control survey. Participation in placing targets, extracting data and formatting extracted data to the client’s specifications can speed up the survey and ensure a cost-effective result.

Table 2 summarises how survey firms and organisations can participate and become involved in MLS surveys.

Table 2: Possible actions to become involved in MLS surveys.

	Action	Description
1	Check control – horizontal and vertical	Complete the control survey – find marks, digital level. This major task includes working with traffic management.
2	Target placement	Place appropriate targets along the corridor – whether with physical markings or with TLS.
3	Provide validation points	Conduct an independent QQ string survey at identified cross sections across the road. This will satisfy you and the end user customer that the work has met accuracy specifications.
4	Scan and collect data – GNSS base stations	Assist the scanning process. Provide base stations on control marks. Assist the scanning crew.
5	Extracting data from the point cloud	Participate in extracting the data from the point cloud – this requires software and trained personnel. Those familiar with current TLS systems are well placed to do this.
6	Delivering the data in the correct format for customer	If you are familiar with the correct client formats and data protocols, you are able to finalise the datasets before delivery – to ensure the customer receives data in the correct format.

CONCLUDING REMARKS

Mobile Laser Scanning has made an impact on corridor surveys for road and rail. The technology continues to develop and improve, and there will be an increasing market demand for high-accuracy MLS data. New applications are being developed so that point clouds can be used for a range of applications other than survey (e.g. visualisation, planning and design).

There is a high barrier to entry to join in this market as significant investment is required in hardware and software. Also vendors eager to sell equipment often do not mention the high cost in training and maintenance required to run equipment. All MLS surveys, however, include a significant amount of traditional survey measurement. These tasks are outlined above and include control surveys, placing targets, feature pick-up, data extraction and data formatting.

It is worth for survey companies and organisations to consider the use of MLS for future corridor surveys. In order to be involved, they should contact their MLS supplier to discuss the job requirements (accuracies and deliverables), the costs involved and the level of participation of their own organisation. Working together with your MLS data supplier will provide the best opportunity to deliver high-accuracy corridor surveys to your client, within their specifications quickly and cost-effectively.

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Use of Spatial Data During and After the Blue Mountains Bushfires in October 2013

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ABSTRACT

This paper addresses aspects of the devastating bushfires in the Blue Mountains in October 2013, which resulted in the destruction of 197 homes across the Council area. In total, 186 homes were destroyed in Winmalee, nine in Mt Victoria and two in Mt Irvine. Many other homes were damaged. The houses in Winmalee and Mt Victoria were destroyed in the first couple of hours on the first day. The initial fires were devastating in intensity and speed, which greatly impacted the community in a short period of time before the fire services could adequately react. This paper outlines the behaviour and extent of the actual fires (showing the mapping of the fire over its stages), the extent of the damage caused by the fires with homes lost and utilities services destroyed, and the immediate response by Council and the State Government in the aftermath of the fires. The Survey and Design Section of Blue Mountains City Council was involved in ground-truthing of conflicting damage data being received from various agencies. Initial data was received from NSW Fire and Rescue who were on the scene during the fires. Some of this data was conflicting with other reports and Council needed to rapidly know the true situation. Council staff drove around the areas affected by fire, mapping damage onto a tablet with Council cadastral data on MapInfo. This was cross-checked against the many damage reports and the information corrected as required. The data was downloaded into the Council systems each day. Photographs were taken of each damaged or destroyed house and aerial photographs were obtained and utilised in this process. The immediate response included many other sections of Council, State Government and the Army. Works included assessment of damage to Council parks/reserves (including structures, trees and catchment impacts), issues with asbestos and other contaminants and assessment of damage to all service utilities. Immediate works were undertaken to protect stream catchments from impact of erosion and contamination from run-off in the event of rain. The paper also addresses the responses that were needed under the State Recovery Plan and its implementation through the Local Council.

KEYWORDS: *Bushfires, impact, disaster response, mapping*.

1 INTRODUCTION

The Blue Mountains have always had a close relationship with bushfire. Over the last 20 years, fires have increased in both frequency and intensity as the summers are discernibly more conducive to intense fire activity. Between 16 and 29 October 2013, the Blue Mountains were devastated by three major wildfires. Such fire activity with three major fires burning simultaneously was unprecedented and required a major response by all fire services, under overall control of the NSW Rural Fire Service (RFS), many State Government departments and agencies, Blue Mountains City Council (BMCC), Lithgow Council and the community of the Blue Mountains.

These fires triggered a declaration under Section 44 of the Rural Fires Act 1997. A Section 44 declaration is used when the RFS Commissioner declares that a fire cannot be managed without drawing in extensive resources from other areas. This declaration was in place between 17 October and 29 October 2013. The Prime Minister (Hon. Tony Abbott) and the NSW Premier (Hon. Barry O'Farrell) announced a Natural Disaster Declaration for the Blue Mountains City and Lithgow Local Government Areas on 19 October. A State of Emergency, under Section 33 of the State Emergency and Rescue Management Act 1989, was declared by the Premier on 20 October 2013. This declaration provided emergency services with additional powers to undertake additional safety measures as required and was lifted on 30 October (BMCC, 2014).

This paper provides an overview of the extent of the fires and describes the damage caused by the fires in both housing losses and infrastructure. It then outlines aspects of the response required within the Blue Mountains City Council in ground-truthing reports of property damage and investigation of other damage within their area of responsibility. Finally, the paper provides an overview of the recovery process, which was developed in the aftermath.

2 THE FIRES

2.1 Extent of the Fires

Rarely have the Blue Mountains seen such a large proportion of its area under such a state of intense fire, with three separate fires burning at the same time. This meant that the total fire area had many different fire fronts to be contained and there were variable intensities and fire conditions across the mountains to be taken into account in the planning to fight and contain these fires. All the fires were driven by high temperatures, strong winds and low humidity, generating high intensities and a high rate of spread, which made their containment extremely difficult. Figure 1 illustrates the extent of all fires in October 2103.

This was a tremendous challenge to the firefighting effort and an unprecedented situation for the Blue Mountains community. Most of the fire damage occurred in the first few hours after the start of the Linksview Fire and the Mount York Fire, respectively. In this initial period, all firefighting responses were focused on protection of life as first priority and protection of property as secondary priority. As the situation became apparent and more resources in personnel and equipment were deployed, the focus was able to be moved fully onto property protection and the containment of the fires.

2.2 State Mine Fire

The State Mine Fire ignited on 16 October 2013 following live ammunition firing on a military range just to the north of Lithgow. It could not be brought under control initially, while in the military firing range, because of the high possibility of live ammunition. It could also not be contained when it spread outside the firing range.

This fire was the largest of the fires and spread to the east. The main threats to townships were at Lithgow, Clarence, Dargan, Bell, Mt Wilson and Mt Irvine. The fire burnt approximately 56,000 ha, destroying two homes (Mt Irvine and Mt Wilson), significantly damaging two others and destroying three outbuildings.

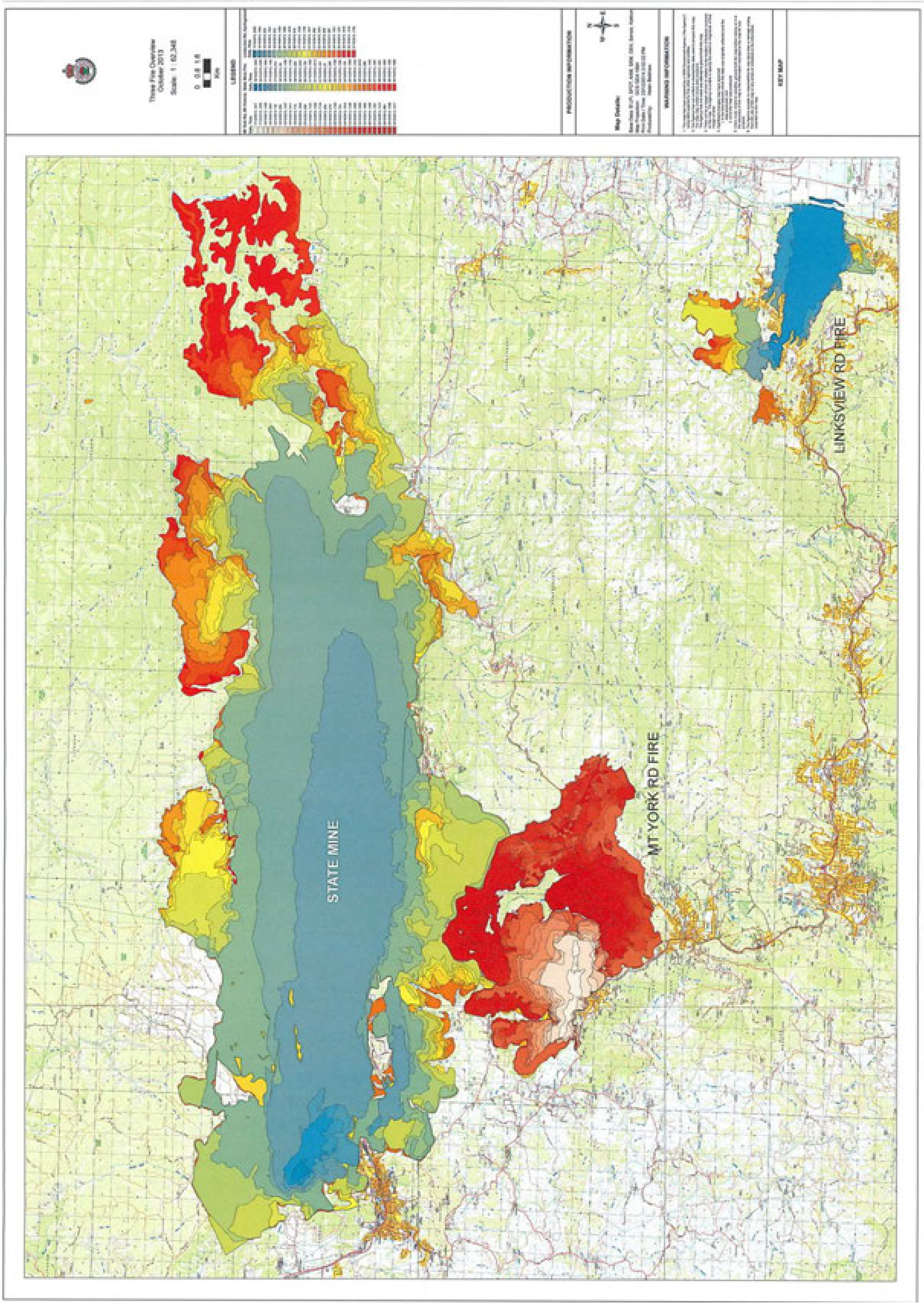

Figure 1: Map of Blue Mountains showing the extent of all fires in October 2013.

2.3 Mount York Fire

The Mount York Fire ignited on 17 October 2013 in the vicinity of Mt York Road at Mt Victoria. It was believed to be started by a fallen high-voltage power line. This fire was centred around Mt Victoria. Fire intensities were greatest on the first afternoon when the property damage was incurred around St Georges Parade to the Darling Causeway. This fire burnt approximately 8,400 ha. As a result of planned burning, this fire joined up with the State Mine Fire, which simplified the number of fronts in the fire and hence helped in its containment. It destroyed nine homes, significantly damaged one house and destroyed other outbuildings.

2.4 Linksview Road Fire

The Linksview Road Fire was the most devastating of the three fires, particularly on the first day. It ignited on 17 October 2013 in the vicinity of Linksview Road. It was ignited by sparking from a domestic power line to vegetation. It burnt quickly to the east on a fairly narrow front with high intensity and devastating effect, crossing Hawkesbury Road and burning towards Yellow Rock, generally along Singles Ridge Road (Figure 2). This fire burnt approximately 3,600 ha, and the worst hit streets (among many streets which suffered) were Buena Vista Road, Emma Parade, Heather Glen Road and Yellow Rock Road. The fire destroyed 186 homes, significantly damaged 132 homes and destroyed 93 other outbuildings.

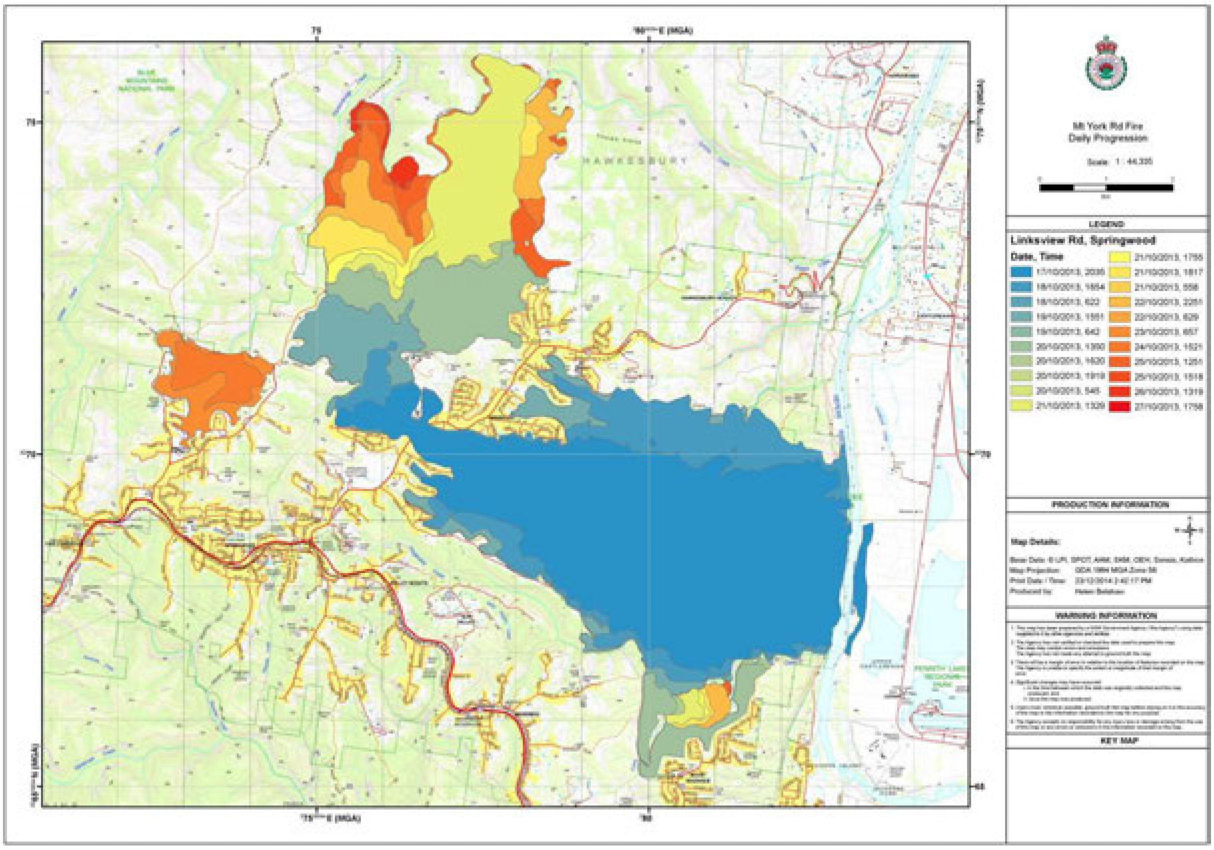

Figure 2: Extent of the Linksview Road Fire.

3 THE NEED FOR GROUND-TRUTHING OF FIRE DAMAGE DATA

3.1 Initial data

Council was the central data receiving organisation for all issues of damage to housing and emergency recovery works in the immediate aftermath of the bushfires. An Initial Impact Assessment of the Linksview Fire was compiled by the Environment Protection Authority (EPA) and provided to Council. This consisted of a series of maps of the Linksview Fire using aerial photography, showing the initial assessment of homes destroyed (Figure 3).

Figure 3: EPA aerial map of initial assessment of damage.

Initial damage reports were compiled by NSW Fire and Rescue and the Department of Public Works. These reports were quite detailed and in their standard formats. They formed the basis of the initial Bushfire Damage Register developed by the Development and Customer Services Group in Council, and particularly by the Building and Compliance Services Section and the Development and Planning Services Section (Figure 4). These sections were primarily involved in the collation of the information on fire damage to buildings and associated structures, so as to adequately respond to a flood of enquiries in relation to a myriad of community concerns and issues.

Subsequent to these initial assessments, there was a constant flow of information over the coming days into Council from affected residents and from Council staff, State Government (primarily Public Works) and the Defence Forces and other service authorities and agencies who were involved in the immediate recovery works. At times, there was difficulty in reconciling the initial data with the subsequent reporting and information flow. Council had concerns as to the overall accuracy of the Damage Register being developed, so it was decided to undertake a process of ground-truthing of the data in the Register, and add and develop the Register as required.

The problems in the original data were probably due to:

- The initial information being collected under emergency conditions where much of the ‘normal’ physical features have just disappeared, particularly with the normal property identifiers (e.g. letter boxes, identifiable house numbers, fences) being destroyed in many areas. This occurred particularly where there was a mix of vacant land and destroyed housing with no visible boundaries between them apparent in the burnt landscape.
- Inconsistency with house numbering in some streets, particularly near intersections or houses with double frontages, or ‘battle-axe blocks’ with accesses through rights of way.

In some cases, this resulted in particular homes being reported destroyed when they were not, and others being reported as okay when they were not.

BushFire 2013 Register

Registe Key	Property Address House	House	House End	Street	Suburb	Rapid Damage Assessment Status	Damage	Posting Status	Date	by	ID	Note	Structural Present	Rectified	Tree Hazzards Present	Rectified	Asbesto Present	Pool Present	Damage	Owner Comment
131	4		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Unsafe	10/22/2013		RDA6-14	Totally Destroyed	Yes				No			we were unable to attend the meeting, our son attended on our behalf (& in his own right) and collected information for us.
132	6		0	Heather Glen Road	YELLOW ROCK	Completed	0-25%	Inspected fit	10/23/2013		RDA6-15	Solar power damaged and isolated by owners	No		Yes		No			
134	8		0	Heather Glen Road	YELLOW ROCK	Completed	0-25%	Inspected fit	10/23/2013		RDA6-16	1. Structurally sound. Minor damage to lattice. 2. 6000 litre water tank at rear intact.	No				No			
324	9		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA6-19		Yes				No			Garbage bin replacement, Green waste, dangerous quantaties CSR 185426
136	10		12	Heather Glen Road	YELLOW ROCK	Completed	0-25%	Inspected fit	10/23/2013		RDA6-17	1. Minor fire damage to roof area and bedroom. 2. pool intact but fencing TDBF.	No				No			
325	11		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA6-20	LPG IN SECTOR C	Yes		Unconfirmed		No			Building is a total loss. Please leave the retaining wall on the site. Did not feel there was asbestos on site. Not giving consent at this stage, but will call asap if need be. Army there on Sunday removing trees. Verbal consent given at the time. 5/11/20
138	14		16	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Unsafe	10/23/2013		RDA6-18	Structure including vehicle and two sheds TDBF.	Yes				No			If rebuild what is process - will fees and charges be waived. Land enquries
326	15		0	Heather Glen Road	YELLOW ROCK	Completed		Inspected fit	10/24/2013		RDA6-21	SECTOR A POOL CHLORINE AND DEAD TREES PARTIALLY BURNT THROUGH.	Yes		Yes		No			
183	18		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA5-29		Yes				No			Would like ADF to help clear the fire damaged trees not covered by insurance. The trees have been knocked over as were in imminent danger of falling. Told customer this may not be possible under the guidelines ADF are working to.
185	20		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA5-30		Yes				No			House destroyed
186	22		24	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA5-31		Yes				No			Fully destroyed. Septic tank on site
188	26		0	Heather Glen Road	YELLOW ROCK	Completed		Inspected fit	10/24/2013	PW	RDA5-32 - PW-26	compressed fibro cement board of outbuilding. Probably asbestos. Friable bc its broken up. Requires bonding with pva and removal. PW Comment: House destroyed	Yes				Yes			
327	27		29	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA6-22	SECTOR C POOL FENCE DAMAGED DUE TO SUBSIDENCE.	Yes		Unconfirmed		No			Structural damage to dwelling and trees
330	28		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA6-24	SECTOR A 1 X LARGE GUM TREE ACETYLENE,ARGON AND OXYGEN CYLINDERS.	Yes		Yes		No			
190	30		0	Heather Glen Road	YELLOW ROCK	Completed	None	Inspected fit	10/24/2013		RDA5-33		No				No			sheds, garden 2 x grow houses destroyed.
329	31		33	Heather Glen Road	YELLOW ROCK	Completed	None	Inspected fit	10/24/2013		RDA6-23		No				No			
191	34		0	Heather Glen Road	YELLOW ROCK	Completed	75-100%	Inspected fit	10/24/2013		RDA5-34	gate or barricade as a matter of urgency due to children next door.	Yes				No			Please do not contact regarding the bushfires. Please contact husband.
102	35		37	Heather Glen Road	YELLOW ROCK	Completed		Unsafe	10/23/2013		RDA5-28		Yes				No			

Figure 4: Example of initial collation of data by Council to form the Damage Register.

3.2 Procedures Adopted

3.2.1 Responsibilities

A co-ordinated whole of Council approach was required to respond to the information and data requirements to allow an adequate response to the many and varied needs for disaster planning and the following recovery process. By late October, it was apparent that the Council needed to resolve the issues with the various flows of information coming into Council – to verify the accuracy of the Damage Register and to establish accurate mapping layers on the corporate database and mapping systems. The mapping needed to show fire impacted properties, areas of public safety concerns, areas of environmental concerns and other relevant and essential information.

The Building and Compliance Services Section and the Development and Planning Services Section within the Development and Customer Services Group co-ordinated the data collection and mapping as they were the focus for information on a wide variety of issues, such as:

- Issues of public safety in the fire damaged areas on both public and private lands. Immediate issues were the presence, and possible impacts, of asbestos in damaged and destroyed houses, the safety of many swimming pools which remained after fencing and houses were destroyed, and damage to on-site septic systems with melting of tank lids.
- Development and building enquiries in relation to the repair and reconstruction of damaged and destroyed houses. These were being asked at an early stage by landowners and insurance companies.
- Presentation at a series of public meetings, which were held regularly in the fire damaged areas to keep residents and affected people up to date with the situation.

The Environmental Sustainability Group were heavily involved in public safety of dangerous trees still standing, particularly near private land or along walking tracks, and in the assessing of damage to facilities in parks and reserves. They were also heavily involved in immediate environmental concerns in the fire areas and clearing of fallen trees on roads and access ways.

The IT section established the IT protocols and systems to collate the information. The Geographic Information System (GIS) staff developed the mapping workspaces on MapInfo and validated the information and surveyed data as it arrived. This was a continuous process and data was being regularly updated on the corporate systems. Blue Mountains City Services required the mapping so as to respond in their many areas of operation in the immediate recovery works, such as clearing trees, cleaning up and waste removal.

3.2.2 Equipment Used in Survey

Most of the data was collected with a motion tablet. This tablet ran with MapInfo v11.5 software, which was loaded with a custom workspace for data capture and logging, and included the Council aerial photography layer, cadastral overlays, street numbering and other relevant data layers. This is enabled through an editable layer and data was captured through Mapbasic scripts and forms writing to a customised table. The tablet had an inbuilt GPS receiver, which interfaced with the MapInfo software through a tool, Blue Marble Geographic Tracker, which logged the geo-coordinates from the GPS and then mapped them spatially in MapInfo.

The survey for ground-truthing of the housing damage register was done mainly by car, using the inbuilt GPS receiver in the tablet. The inbuilt GPS was generally accurate to 2.5 m in good conditions, which was fine for this purpose. As we drove around the streets, our position was tracking on the MapInfo layers and aerial photography in the tablet. Hence there was no confusion over the identification of the individual residential properties, no matter what the ground conditions were.

The surveys for the trees and vegetation mapping were carried out with these same tablets. However, the positional accuracy was enhanced with a Trimble external Pro6H antenna, which had Bluetooth connectivity to the tablet (Figure 5). This delivered sub-metre accuracy to the survey, which was more than sufficient for the purpose.

Figure 5: Survey equipment (left) and survey with external antenna (right).

3.2.3 Protocols and Field Procedure

The data-truthing survey was conducted by car due to the extent of the work and travel involved. Every property in the fire affected areas was visited and visually assessed from the road or driveway. This meant inspecting and identifying every property in the Springwood-Winmalee fire area, Yellow Rock, Mt Victoria fire area, Mt York, Bell, Mt Wilson and Mt Irvine (west and north side areas). No buildings were entered or closely inspected because of safety concerns and time constraints. A database of property damage related to property identification was required as soon as possible.

All buildings were assessed in four categories:

- Zero – no damage.
- Partial – damage present but habitable.
- Substantial – damage present but not habitable.
- Total – destroyed.

At every property, we:

- Identified the property on the tablet GIS. Our position on the MapInfo aerial photography and cadastral overlay was shown at all times with an icon.
- Gave a visual assessment of the property and entered the assessment directly against the property ID in the MapInfo workspace. Some notes could be made here as well.

- Photographed the property, without being invasive to any residents.
- Checked our assessment against a printout of the existing register of damaged properties and either verified or noted amendments.
- Made note of properties that suffered damage or were destroyed but were not noted on the property register. By this cross-checking of the register, it soon became apparent which properties were wrongly identified by address.
- Took note of any other information, such as resident’s concerns, their well-being, or noting further information that was relevant.

The tablet was downloaded onto the Council network at convenient intervals during the survey, generally every second day. This time period was adopted so that the data could be assessed by relevant council staff for consistency and adequacy of the results. If any changes were required, or more data collected, this could be done in a timely fashion while we were still in that immediate area. Council’s GIS staff then edited the data and merged it into the corporate database and mapping system. They also added other verified information to the database that was reported by other sources.

The photographs (Figure 6) were downloaded at the same time and identified by property address. These were then linked to the MapInfo workspace and to the Damage Register by property ID (Figure 7).

During this work there was a lot of contact with the community in the fire damaged areas. There were a surprising number of people who remained living in these areas, even though there were very limited services. Water was generally available but little or no power, gas or telecommunication services. Many had stayed in their homes and fought the fires, and some others had moved back when their homes were found to be safe.

Figure 6: One of many houses destroyed.

Figure 7: Survey working map.

Access to these areas was restricted to residents only and recovery workers, with road blocks at main entry points off Hawkesbury Road. These residents were very effective in keeping anti-social behaviour out of the area. People without a purpose for being there, who had talked their way past the road blocks were generally questioned and discouraged from being there. This included opportunistic tourists and photographers. The police also had a continual roving presence and contact was maintained with them.

A considerable amount of time was taken talking to residents about what we were doing and why. The response was always positive, and we often learned extra information about problems and issues that they faced which we were able to report back to the relevant people.

3.3 Aerial Photographs to allow Checking of Data

In order to assist in the data verification process, Council needed to access aerial photography post fires to get an overview of the fire areas. To this end, aerial photography from a private company, Skyview Aerial Photography, was commissioned. The photography was not ortho-rectified but was of high resolution and covered all fire areas and environs. The photography was of a panoramic nature and not of individual houses, but was more than capable for our purposes, particularly in being used for data verification purposes. Council later received post-fire high-resolution, autho-rectified imagery from the Emergency Information Co-ordination Unit of Land and Property Information (LPI) as well. Both these sets of photography were invaluable to us in the data validation process (Figures 8-10).

Figure 8: Linksview Fire.

Figure 9: Mount York Fire.

Figure 10: Pattern of damage.

3.4 Results

The staff at Council continually developed its own Register of Fire Damage to properties, in which it was fully confident of the accuracy of the data. This Register was the hub for all information relating to the fire damage with links to all pertinent information locations and its sources (e.g. NSW Fire and Rescue, Public Works, Council, residents, Defence Forces), relevant photographs and the MapInfo workspace. It was a live document, and information was updated as recovery works proceeded.

A MapInfo workspace was developed with a great deal of information attached graphically to each property in the fire areas. For each property, various attributes were linked and presented by symbols on each property in the workspace.

Graphical information shown for each property was (Figure 11):

Damage to the homes (see section 3.2.3):
- Zero – home not damaged.
- Partial – home damaged but habitable.
- Substantial – home damaged but not habitable.
- Total – home destroyed.

Other considerations in the damage risk of the property:
- Inspected fit for habitation.
- Structural issues.
- Unknown, to be checked.
- Presence of asbestos.
- Restricted, e.g. pools and septic.
- Whether the property had been made safe.

The access of the data (both Damage Register and graphic workspace) was available corporately to all staff working on the fire recovery process, i.e. the Recovery Team and associated agencies, which had been established in Springwood offices to deal with the recovery process. In addition, the information was made available to the many agencies that were working to reconstruct infrastructure in the area. Figure 12 shows part of the updated Property Damage Register.

Figure 11: Part of MapInfo workspace with graphical information (aerial underlay turned off).

Reference Number	Property	ID No.	Structural Hazzards	% Building Damage	Asbestos present	Posting status	F&R NSW comment	Swimming Pool Present	Inspected by	Tree Hazzards	Swimming Pool Damage	O/S Damage
2	46C Birdwood Avenue, WINMALEE NSW 2777	RDA1-1	Y Yes	L5 75-100%	N No	U Unsafe			FR			
3	46A Birdwood Avenue, WINMALEE NSW 2777	RDA1-2	Y Yes	L5 75-100%	N No	U Unsafe			FR			
4	49 Birdwood Avenue, WINMALEE NSW 2777	RDA1-3	Y Yes	L4 50-75%	Y Yes	U Unsafe			FR			
5	156 Hawkesbury Road, SPRINGWOOD NSW 2777	RDA1-4	Y Yes	L5 75-100%	Y Yes	U Unsafe		Y Yes	FR			
6	158 Hawkesbury Road, SPRINGWOOD NSW 2777	RDA1-5	N No	L5 75-100%	N No	U Unsafe			FR			
8	425-443 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-22	N No	L1 None	N No	F Inspected fit		N No	FR			
9	160 Hawkesbury Road, SPRINGWOOD NSW 2777	RDA1-6	Y Yes	L5 75-100%	N No	U Unsafe		N No	FR			
10	295-299 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-23,RDA4-46,RDA6-55	Y Yes	L5 75-100%	N No	U Unsafe		Y Yes	FR			
11	173 Hawkesbury Road, WINMALEE NSW 2777	RDA1-7	Y Yes	L3 25-50%	Y Yes	R Restricted		N No	FR			
12	411-413 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-24	N No	L1 None	N No	F Inspected fit		N No	FR			
13	407-409 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-25 -PW-03	Y Yes	L1 None	Y Yes	F Inspected fit		N No	FR.PW			
14	32 Emma Parade, WINMALEE NSW 2777	RDA3-7	Y Yes	L5 75-100%	Y Yes	U Unsafe	Structure completely destroyed by fire. External walls still standing. No floor or roof remains.	N No	FR			
15	399-405 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-26, RDA4-23	Y Yes	L5 75-100%	Y Yes	U Unsafe			FR			
16	40 Emma Parade, WINMALEE NSW 2777	RDA3-8	Y Yes	L5 75-100%	Y Yes	U Unsafe	Structure completely destroyed by fire. 1 external wall remaining. Large amount of friable asbestos over entire debris pile.	N No	FR			
17	340-360 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-27 - PW-04	Y Yes	L1 None	Y Yes	F Inspected fit			FR.PW			
18	171A Hawkesbury Road, WINMALEE NSW 2777	RDA1-8	N No	L2 0-25%	N No	F Inspected fit	window damaged and partial ceiling due to fire and water.	N No	FR			
19	393-397 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-28 - PW-05	Y Yes	L5 75-100%	Y Yes	U Unsafe		Y Yes	FR.PW			
20	3 Colville Road, YELLOW ROCK NSW 2777	RDA1-10		L1 None	N No	F Inspected fit	Fence and Shed damaged in rear yard ,loose sheets may be a hazard in high wind	N No	FR			
21	379-383 Singles Ridge Road, YELLOW ROCK NSW 2777	RDA4-29	N No	L1 None	N No	F Inspected fit		N No	FR			
22	5 Colville Road, YELLOW ROCK NSW 2777	RDA1-11	N No	L1 None	N No	F Inspected fit	damage to rear boundary fence and lhs fence		FR			
23	87 Buena Vista Road, WINMALEE NSW 2777	RDA5-1	N No	L2 0-25%	N No	F Inspected fit	damage to retaining walls and front pergola. Loose timbers from damaged pergola needs to be pulled down.	N No	FR			
24	7 Colville Road, YELLOW ROCK NSW 2777	RDA1-12	N No	L1 None	Y Yes	U Unsafe	Dangerous trees and limb will need to be removed before asbestos is dealt with as they overhang work area		FR	Y Yes		
25	83 Buena Vista Road, WINMALEE NSW 2777	RDA5-2	Y Yes	L5 75-100%	N No	U Unsafe	TOTAL DESTRUCTION OF ALL PROPERTY PRESENT AS PER IMAGE. HOUSE AND VEHICLE		FR			
26	57 Emma Parade, WINMALEE NSW 2777	RDA3-9	Y Yes	L5 75-100%	Y Yes	U Unsafe		N No	FR			
27	79 Buena Vista Road, WINMALEE NSW 2777	RDA5-3	N No	L2 0-25%	N No	F Inspected fit	DAMAGE TO FRONT DECKING, GARDEN SHED, REAR VERANDAH	N No	FR			
28	4 Illingworth Road, YELLOW ROCK NSW 2777	RDA3-10	N No	L1 None	N No	F Inspected fit		N No	FR			
29	64 Buena Vista Road, WINMALEE NSW 2777	RDA5-4	Y Yes	L5 75-100%	Y Yes	U Unsafe	TOTAL DAMAGE. ASBESTOS PRESENT	N No	FR			
30	9 Colville Road, YELLOW ROCK NSW 2777	RDA1-13	N No	L1 None	N No	F Inspected fit	1. pre-existing minor hazard from trees 2. minor damage to lhs fence	N No	FR	Y Yes		
31	6 Illingworth Road, YELLOW ROCK NSW 2777	RDA3-11	N No	L1 None	N No	F Inspected fit		N No	FR			
32	5 Sunny Ridge Road, WINMALEE NSW 2777	RDA2-1	N No	L1 None	N No	F Inspected fit	lopping of large gum trees at rear of residence	N No	FR	Y Yes		

Figure 12: Part of updated Property Damage Register.

4 OTHER IMMEDIATE RESPONSE WORKS BY COUNCIL

4.1 Survey of Dangerous Trees on Public Property

Due to issues of public safety, it was necessary to inspect and make safe all public areas, parks and reserves, walking tracks, camping areas and the interface between urban and bushland areas, which were managed by Council. This was a huge task in the Blue Mountains, with large areas of bush and urban interface. Council only mapped the areas under the care and control of Council.

All these areas were walked, and qualified staff inspected and assessed every tree that could have an impact on public safety. If a tree required attention, it was numbered, photographed, surveyed by backpack-mounted GPS and mapped onto the MapInfo workspace in the tablet (Figure 13). Trees were assessed as high or medium priority for works.

High priority works included trees that:

- Had fallen and were blocking roads and accesses.
- Were standing but damaged and had potential to fall.
- Had limbs that could fall.
- Had potential for causing injury to people or further damage to nearby property.

Medium priority works included trees that had not fallen but were not in immediate danger of doing so, and trees that had already fallen and needed to be cleared or made safe. Areas of rapid weed regrowth were also becoming apparent, so these were mapped as well for further treatment.

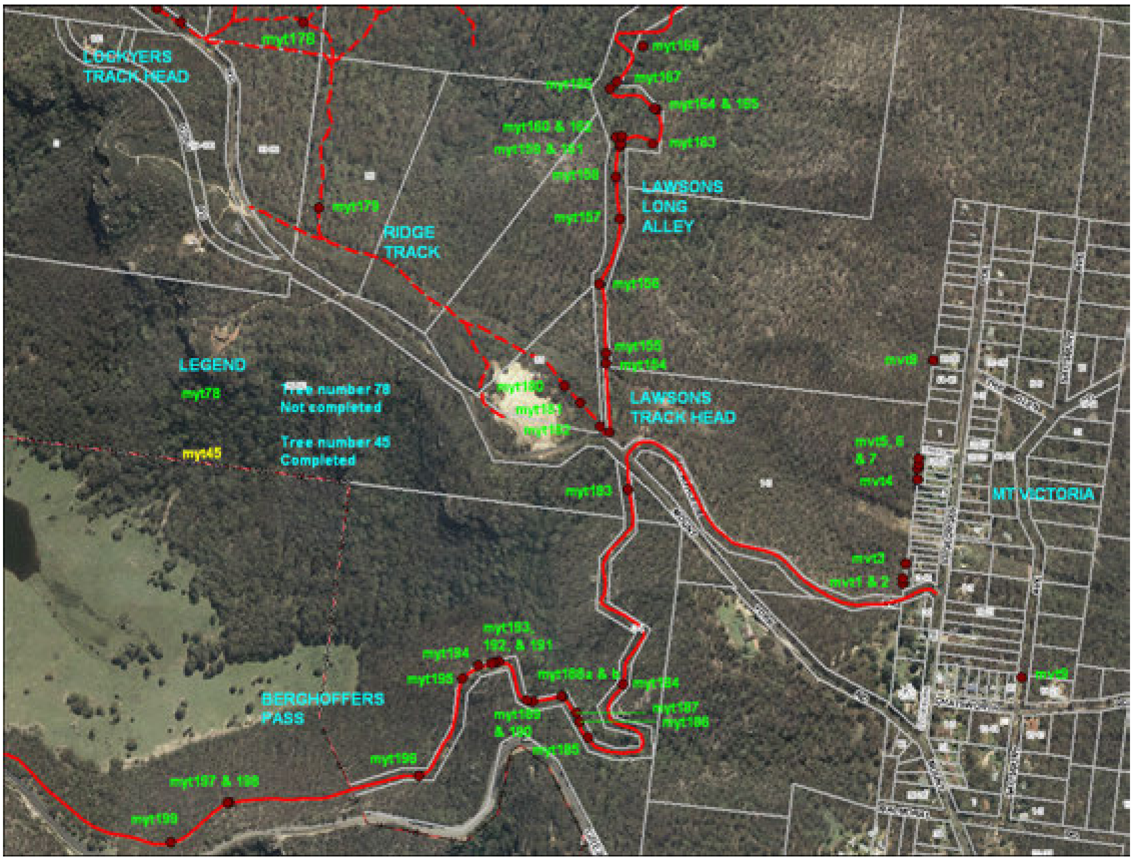

Figure 13: Plan of dangerous trees at Berghoffers Pass and Mt Victoria.

After the mapping was completed, contractors were engaged to clear the trees. In areas that had reasonable access, the trees were cut, logged and chipped. These chips were then used in other parks and reserves where needed. In areas of no access, the trees were generally logged up and then moved further into the bush. This was a practical measure that needed to be undertaken, although the volumes of timber in this situation were not large.

4.2 Environmental Concerns from Drainage

A high level of concern emerged with the stability of the soil after the fires burnt all the ground cover. These burnt areas had exposed top soil, which was very fine and friable. There was high potential for erosion and contamination issues in the event of rain. Erosion was possible as sheet erosion across slopes and concentrated erosion in drainage channels and watercourses (Figure 14). Contamination of the bush soils was possible from contaminants being washed from properties that were damaged or destroyed.

A range of sediment and run-off control devices were put in place immediately by Council staff and the Defence Forces. Such devices included (Figure 15):

- Sandbags and filter geofabric over all drainage pits. This was aimed at preventing contaminants and silt from washing into the bush.
- Utilising kerbing as detention basins in low points. This was quite effective in one cul-de-sac, where the drainage pits were blocked and the kerb levels were raised with coir logs.
- Installing sedimentation fencing and sand bag reinforcement at drainage outlets.
- Installing coir logs along drainage channels to form small retardation dams to reduce the velocities of run-off. These were located between 10 and 20 m intervals along watercourses depending on the gradients of the drainage channels.
- Installing coir logs at areas of possible contaminant flow from damaged properties, such as across driveways and hardstand areas.

There were periods of rain following the fires, but no high-intensity storms. After each day of rain, all the structures were inspected and strengthened or adjusted if necessary. Street sweeping trucks were also continuously circulating, vacuuming the roads of sediments and contaminants. This was a key to the effectiveness of the measures that were put in place.

Figure 14: Typical drainage channel (left) and typical drainage line (right).

Figure 15: Typical treatment drainage pits (left) and typical treatment hardstand areas (right).

4.3 Public Safety Concerns

4.3.1 Asbestos

One of the critical safety aspects to be dealt with in the fire areas was the presence of asbestos in many of the homes that had been destroyed or damaged. It presented a safety issue for windborne particles, for people accessing the properties and for run-off during rain into the surrounding properties or bushland. Consequently, all houses were inspected by Public Works and those found to have asbestos were sprayed with a binding agent until they could be safely removed. The blue binding agent was quite noticeable throughout the fire area (Figure 16).

Figure 16: Houses with asbestos.

4.3.2 Backyard Swimming Pools and Other Hazards

Another major issue for public safety was the many swimming pools that were constructed at houses which were badly damaged or destroyed. This resulted in easy access to many of these pools after the fencing was badly damaged or destroyed. Some pools were full of water, though very dirty, and some had very little water. All houses in the fire area with pools were inspected for safety compliance and cross-checked with Council's existing pool records.

In addition, the general property damage inspections highlighted other hazards presenting a possible danger to people in the area. On some properties with on-site septic systems, the plastic tanks were compromised with their lids melting in the fire. This presented a safety and public health issue as the tanks were sometimes difficult to identify being covered by debris

and ash, and the sewage in the tank was exposed, so was dealt with as a matter of priority. In all cases of safety, particularly with swimming pools, temporary mesh fencing was erected around the hazard until the hazard was made safe.

4.4 Plans of Consolidation: Heather Glen Road

A planning issue arose in Heather Glen Road, which had many destroyed homes. There was a Council planning requirement for Heather Glen Road that imposed a minimum lot size with property development. In general, the minimum lot size was the equivalent of two of the existing lots in the original subdivision and so two lots had to be consolidated with any development. Over time, the street was almost fully developed but several of the properties were never consolidated and the houses were constructed across existing boundaries.

It was decided to ensure that the property consolidations were done with the reconstruction of houses. However, there were concerns that this would be an undue imposition on people at this time. Accordingly, Council approached LPI and obtained a blanket exemption number for compiled plans for the seven affected properties to be consolidated. The offer was also made that Council prepare the Plans of Consolidation for the property owner if they so desired.

4.5 Base Plans Utilised for Other Purposes

Base plans of all the fire areas that Council printed for its own ground-truthing survey were also in demand, both in the immediate fire aftermath and during the recovery process. They were used for a range of information gathering and planning purposes. These base plans were simply cadastral maps with house numbers and street names in either A1 or A3 size.

Two examples of the need for these base plans were:

- Immediately after the fire, they were used by other agencies and contractors, particularly Public Works, to assist in mapping damage that was inspected (such as the asbestos issue) and to locate properties needing particular work. By using these maps, the data was sure to be linked to the correct property ID. Sets of maps were also placed on walls in planning rooms to control the work at properties and mark off when they had been made safe.
- The Red Cross used a wider set of plans incorporating villages and residential areas close to the fire areas. They visited every household in the fire areas (periodically) and the wider area to talk to everyone on their well-being and arranging counselling and other assistance as required. They used the maps for planning the teams of people undertaking this work and to identify those addresses of people requiring more regular assistance.

5 IMPACTS OF THE FIRES ON THE BLUE MOUNTAINS

5.1 Loss of Life and Personal Injury

The Blue Mountains community was extremely fortunate as, despite the large loss of property and the intensity of the fires, particularly on the first day, there was no loss of life. There were some injuries sustained, mainly respiratory issues with smoke inhalation and minor injuries such as lacerations, minor burns and minor orthopaedic injuries. A total of 69 people presented to hospital in the period and 22 people required admission. 12 people were Emergency Services personnel (BMCC, 2014).

5.2 Private Property Damage

As detailed in section 2, the fires were devastating across the Blue Mountains and particularly severe in the Linksview Fire area, affecting parts of Springwood and Winmalee and the whole of Yellow Rock. In the Linksview Fire, 186 homes were destroyed, 132 significantly damaged, and 93 other outbuildings (e.g. sheds) were destroyed and damaged. In the Mount York Fire, which affected the north part of Mt Victoria, nine homes were destroyed, one significantly damaged and several outbuildings were destroyed. In the State Mine Fire, which affected Mt Wilson and Mt Irvine, two homes were destroyed, two significantly damaged and three outbuildings destroyed (BMCC, 2014).

5.3 Essential Services Damage

5.3.1 Water

Approximately $1 million damage was caused to the water infrastructure. Power was lost to quite a number of sewage pumping stations, which necessitated the deployment of generators until power was restored. On-property pressure sewer systems were damaged on 93 properties and 60% needed to be rebuilt, as the tanks and control boxes were melted. It was required to replace over 110 water metres, which had components melt (BMCC, 2014).

5.3.2 Electricity

On 17 October 2013, approximately 5,000 homes were without power services. Within four days, all houses were reconnected to power services except for those homes destroyed or significantly damaged. Approximately $1 million damage was caused to infrastructure, including cables, poles and five power transformers (BMCC, 2014).

5.3.3 Gas

During the emergency, Jemena disconnected gas supply to 766 customers for safety reasons. Supply was restored in less than two days. However, 182 gas meters were damaged or affected and needed repairs. Gas supplies were disconnected to 98 homes that had been destroyed or significantly damaged (BMCC, 2014).

5.4 Impacts to Council

Council was directly involved with the initial response and recovery phase while trying to maintain its core services. Over 125 frontline staff were involved in the initial response phase and support was required from Penrith and Oberon Councils.

Examples of activities (not previously mentioned in this paper) included:
- Resourcing for the Incident Operations Centre (12-hour rolling shifts).
- Plant and resources in active bushfire response tasks.
- Additional waste collection, cleaning and temporary amenities.
- Additional green waste collection in significantly affected areas and provision of fee-free waste at the tip.
- Traffic control and controlling traffic into/out of the worst affected areas for several weeks.
- Involvement in activities of the Evacuation Centre.

- Supporting the State-led recovery with 15-20 staff assisting with the recovery process before State funding was provided for the establishment of a local recovery team.
- Reduced fees for all rebuilding Development Applications (DAs) and associated costs at both Council and the State Government. This has totalled over $500,000 to date.

By December 2013, Council's contribution translated to more than 16,000 hours of staff time and $680,000 in cost. It should be noted that damage to public property (e.g. signage, seats, bus shelters, guide posts and fencing) caused by the fires was minimal (BMCC, 2014).

5.5 Business Impacts

The business impacts to the Blue Mountains community were substantial, particularly within the tourism and ancillary services sector. The Blue Mountains Economic Enterprise investigated the economic impacts over the Blue Mountains Local Government Area (LGA). They estimated that the total Blue Mountains economic output declined by $71.37 million. The tourism sector in the region (including Blue Mountains, Lithgow and Oberon LGAs) estimated a decline in tourism spending of $66.16 million, a figure which included both day visitors and visitors staying in the area.

These dramatic declines in economic activity affected local employment, direct income to a wide range of businesses and caused flow-on effects in other economic activity throughout the region. Substantial advertising campaigns were required to help address this issue and to get the message out that the Blue Mountains were a safe place to visit and the normal attractions were still in operation (BMCC, 2014).

5.6 Personal Impacts

There were heavy impacts on the immediate community affected by the bushfires, including:
- Fear and trauma for those who lost their homes and had to stay safe during the fires on the first day. The impact of the fires was so rapid that many people could not evacuate the area in time.
- Many people were emotionally exhausted after fighting the fires, either with the RFS or defending their own properties, and being unable to evacuate if they needed.
- Dislocation of the community and services.
- Separation from friends and relatives. Many families had to move out of the area after their homes were destroyed. This meant moving away from their support networks.
- Poor air quality and smoke issues caused many health issues.
- Loss of pets. The RSPCA mapped homes of lost pets to help locate homes of pets that were found and made pets available to those who had lost their own pets.

However, the wider community in the mountains was substantially affected as well:
- Being a close-knit community, everyone knew someone directly affected.
- Most of the community were involved in the bushfires in some way, either through direct action of fighting the fires, indirect help in the firefighting effort or in preparation of their own homes for fire activity.
- Experiencing economic effects and social impacts.

It will take a substantial length of time for many members of the community to recover from this traumatic event (BMCC, 2014).

5.7 Environmental Impacts

With the bushfires being so wide spread, there were substantial impacts on the environment (BMCC, 2014):

- Native fauna suffered greatly in the affected areas. In addition to the more common fauna, it is estimated that 18 threatened species were affected in the Linksview Fire and 21 threatened species in the Mount York and State Mine Fires.
- Native flora suffered greatly although most are fire resilient. Much flora, both trees and understorey, were severely damaged. It is estimated that seven threatened species were affected in the Linksview Fire and 16 threatened species affected in the Mount York and State Mine Fires.
- With much of the flora and understorey burnt, there is a high potential for rapid weed growth, which will threaten natural species.
- Issues of contamination (mentioned previously).
- Issues of erosion and sedimentation of waterways (mentioned previously).

6 DISASTER RECOVERY PLAN

A Recovery Plan was established to drive the recovery process over the next 18 months. It was established to transition from the short-term State-led recovery effort to longer term recovery – consistent with State Government guidelines and resolved by Council. The Recovery Plan is very much a community-based plan, managed by Council with close support from the State Government and including a range of other agencies (Figure 17).

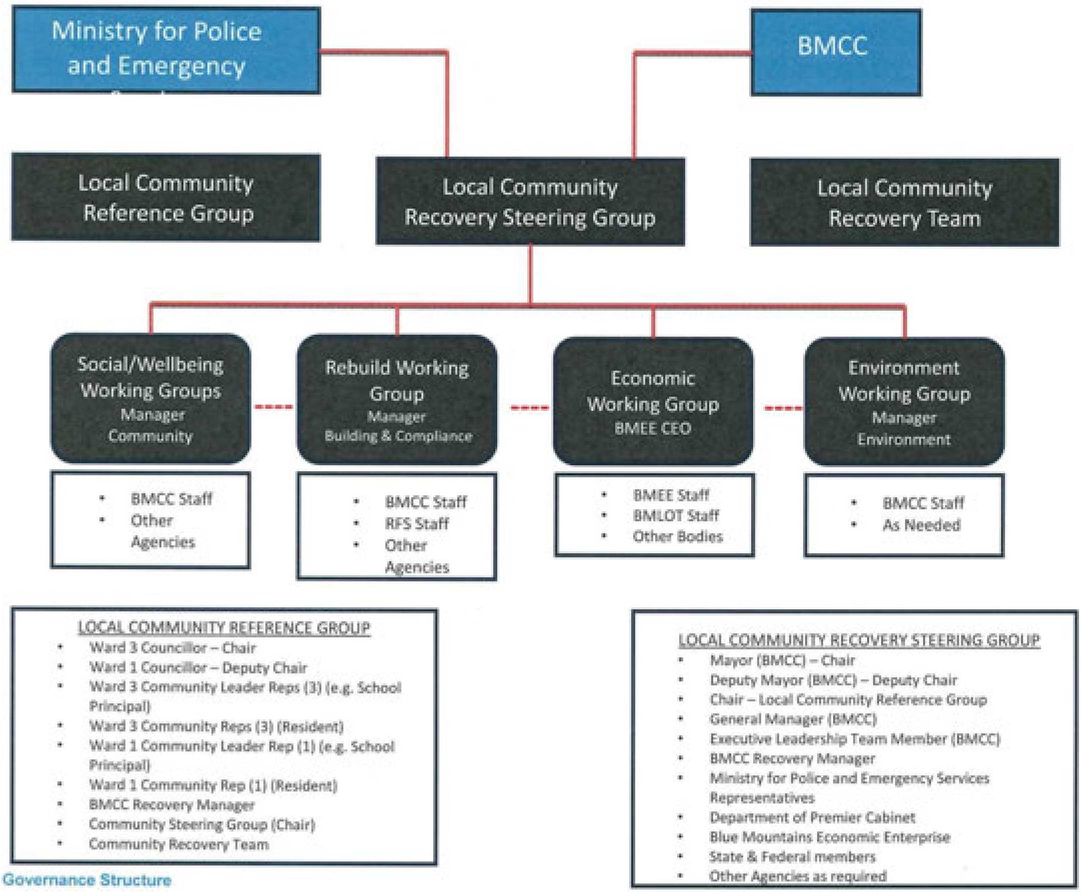

Figure 17: Governance structure, Recovery Team (BMCC, 2014).

The Recovery Plan aims to capture the priority recovery issues and sets out how these will be addressed, why, by whom and by when. This allows for (BMCC, 2014):

- An orderly and systematic way of capturing issues.
- A clear framework for addressing each issue.
- The ability to track the progress and effectiveness of recovery.
- A reporting framework to both the relevant committees and the community.

The objectives of the Recovery Plan are to (BMCC, 2014):

- Provide effective and efficient co-ordination and delivery of programs and services to assist and hasten the recovery of affected communities.
- Co-ordinate programs and services that assist the recovery process.
- Support community involvement and ownership of the recovery process.
- Provide clarity for participating agencies on their responsibilities under the plan.

7 CONCLUDING REMARKS

The major bushfires that occurred in the Blue Mountains in October 2013 were unprecedented in the region and presented a tremendous challenge in fighting the fires and in the recovery process, both in the short and long term. The recovery process is a highly complex process with numerous issues to be resolved and acted upon in a co-ordinated way.

An essential foundation for many recovery activities is an accurate database and associated Damage Register, which is well-constructed, detailed and verified. These are essential to assist in a co-ordinated approach in the recovery process, when there are many separate agencies and contractors involved at the same time.

In this situation, a major effort was required to establish this Damage Register and digital workspaces, detailing as much of the original damage as possible, but also in keeping up to date with the priority recovery works underway by managing information between the responsible agencies.

ACKNOWLEDGEMENTS

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Applying Common Sense to the Definition of an Undefined Public Road

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ABSTRACT

Undefined public roads exist throughout the State. How is an undefined public road defined? The cadastral bible for NSW surveyors, Hallmann's "Legal aspects of boundary surveying as apply in New South Wales" makes short mention of such roads and makes the presumption that the road reserve is centred on the existing road formation. So simply put, the road is where the road is. In other words, the physical road formation as it exists at the time determines the location of the public road reserve. But what if the physical road no longer exists and what if the road has moved significantly from its original position? These are some of the challenges which faced Resource Design & Management Pty Ltd (RDM) when it undertook the acquisition survey for the Upgraded Pacific Highway through the Glenugie State Forest, some 20 km south of Grafton. The purpose of the survey was to establish the additional area to be acquired from the State Forest for the upgrade. Both the Pacific Highway and Lookout Road were undefined public roads and simply relied on a description in the NSW Government Gazette and the Parish Map to establish their status. The challenge was all the more difficult firstly by the fact that Lookout Road had been obliterated by the new works and secondly by the fact that Lookout Road was found to have moved some 500 m from its original position. This paper describes the methodology used, in this instance, to define an undefined public road when the physical road formation no longer exists and also when the road was found to have moved considerably from its original position.

KEYWORDS: *Cadastral surveying, undefined public roads, indirect survey evidence, easement over track in use, GIPSICAM.*

1 INTRODUCTION

The acquisition survey for the Glenugie Upgrade was commissioned approximately two years after the new road works were completed and opened to traffic. The purpose of the survey was to determine the additional land to be acquired under the Roads Act 1993 from the Glenugie State Forest for the Glenugie Upgrade, exclusive of the existing public roads. The upgrade corridor contained two public roads, being the Pacific Highway and Lookout Road.

The survey was complicated as both the Pacific Highway and Lookout Road were found to be 'undefined' public roads. In other words, the roads were public roads that had never been defined by previous surveys. Research into such roads found next to no reference material relating to undefined public roads, apart from Hallmann (1994) and even then there was minimal discussion. Hallmann (1994) makes short mention of such roads and simply indicates the presumption is that the road reserve is centred on the existing formation. Under normal circumstances, where the roads still exist, the survey would have been a relatively simple

matter in so far as determining the location of the formation of the two roads. However, both roads threw up some interesting issues with respect to their definitions which are the subject of this paper.

2 BACKGROUND

The NSW Minister for Planning approved the Glenugie Upgrade project in December 2009. The 7 km upgrade forms part of the larger Woolgoolga to Ballina Pacific Highway Upgrade project. The Glenugie Upgrade extends from Franklins Road to Eight Mile Lane some 20 km south of Grafton (Figure 1).

Figure 1: Location of the Glenugie Upgrade.

The Glenugie Upgrade project is wholly contained within the Glenugie State Forest. The project was completed and opened to traffic in October 2011 (Figures 2 & 3). As the project was wholly contained within the Glenugie State Forest, the acquisition of the corridor was not undertaken prior to the construction works. Resource Design & Management Pty Ltd (RDM) was contracted by NSW Roads and Maritime Services (RMS) to undertake the acquisition survey in June 2013, almost two years after the project was completed.

Figure 2: Glenugie Upgrade under construction looking south.

Figure 3: Glenugie Upgrade opened to traffic.

3 OLD PACIFIC HIGHWAY

3.1 Determining the Road Width

RMS in commissioning the survey advised that some preliminary investigation of the highway through the Glenugie State Forest had indicated the highway was defined as a fixed width on the existing formation. The width was to be confirmed as part of the acquisition survey.

Glenugie State Forest No. 26 was proclaimed by the NSW Government Gazette dated 10 December 1913, following its earlier notification on 2 June 1884. In July 1940, the State Forest was notified in the Government Gazette as National Forest No. 40. The notification related to the whole of Glenugie State Forest No. 26, excluding “a strip of land 3 chains wide embracing the Pacific State Highway passing through this land in a south-easterly direction” (Figure 4).

Arandin or Glen Ugie Creek aforesaid and on all other sides by that bank downwards to the point of commencement EXCLUSIVELY of a strip of land 3 chains wide embracing the Pacific State Highway passing through this land in a south-easterly direction a public road 1 chain wide in a north-westerly direction from the south-western corner of portion 109 Parish of Lanitza

Figure 4: Excerpt from Government Gazette July 1940.

From this notification there was an original expectation that the existing Pacific Highway corridor would simply be 3 chains (60.35 m) wide, embracing the existing road formation. However, a number of road widening plans undertaken by the NSW Department of Main Roads (DMR) in 1939, namely Ms. 3130 Gfn and Ms. 3131 Gfn, showed the adjoining Pacific Highway as being 100 links or 20.115 m wide (Figure 5). This width was also repeated in the subdivision of the adjoining land in 1982 (DP263104).

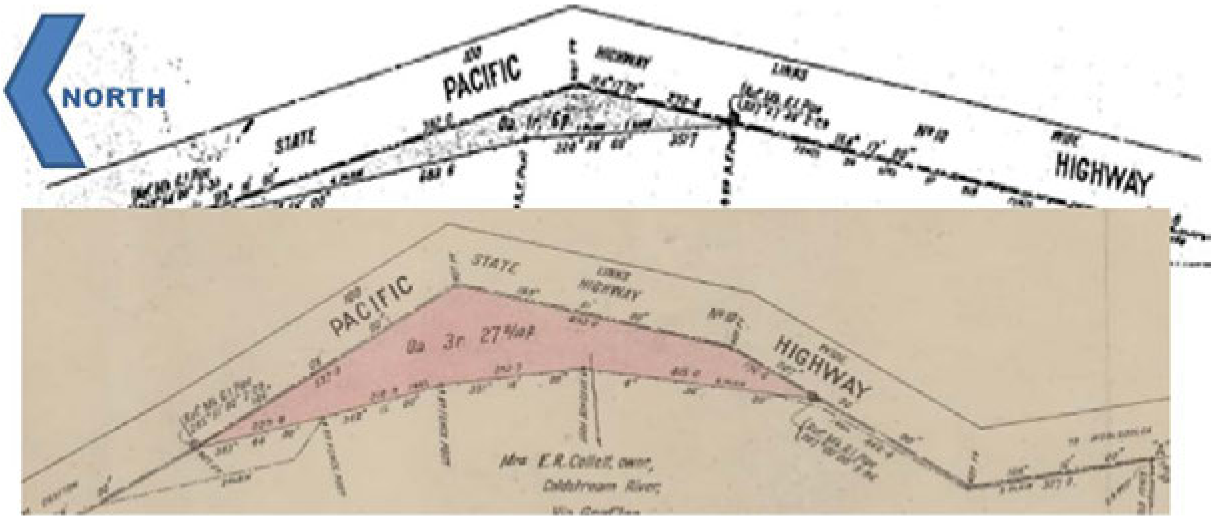

Figure 5: Excerpts from Ms. 3130 Gfn and Ms 3131 Gfn.

Contrary to these plans, DP719173 undertook some road widening of the highway in 1984 and was the first plan of survey required to define the eastern boundary of the highway – it did so on the basis of the corridor being 30.175 m (150 links) wide. The plan also agreed with the width shown on the Parish Map of Lanitza, both of which are shown in Figure 6. Accordingly, the width of the Old Pacific Highway of 30.175 m (150 links) was adopted for the ongoing acquisition survey.

3.2 Defining the Old Pacific Highway

The Old Pacific Highway abuts the eastern boundary of Portions 57 & 58 in the Parish of Lanitza as shown on the Parish Map (Figure 6). However, from near the south east corner of Portion 58 through to Glenugie Creek and beyond, the highway is bordered on both sides by the Glenugie State Forest No. 26. A thorough investigation of this section of the highway did not uncover any previous survey activity or plans to define its location. This lead to the conclusion that the highway was undefined through the State Forest, apart from the area surveyed as part of DP719173 (Figure 6).

Research undertaken into undefined public roads found limited commentary or reference material for such a situation, other than a small mention in Hallmann (1994). Hallmann, in discussion on Section 73 of the Local Government Act 1906, states on pages 6-7, in part, that:

"*Many roads opened through reserves were not defined by survey and only an approximate indication of their position may be obtained from the parish maps. The whereabouts of their boundaries on the ground is a question of fact, the presumption being that the road extends 10.06 metres (33 feet) from either side of the centre line of the prepared surface in use, where the standard width of one chain was dedicated.*

The same section, as amended in 1908, also for the purpose of that Act extended the statutory definition of 'public road' so as to include inter alia a list of roads classified as main roads in Government Gazette dated 31 December 1906, many of which in part were undefined on the ground."

From the above, Hallmann (1994) makes the logical presumption that for an undefined public road, the road corridor extends centrally over the existing formation. On this basis, the centreline of the Pacific Highway where it is bordered on both sides by the Glenugie State Forest would be adopted to define the road corridor.

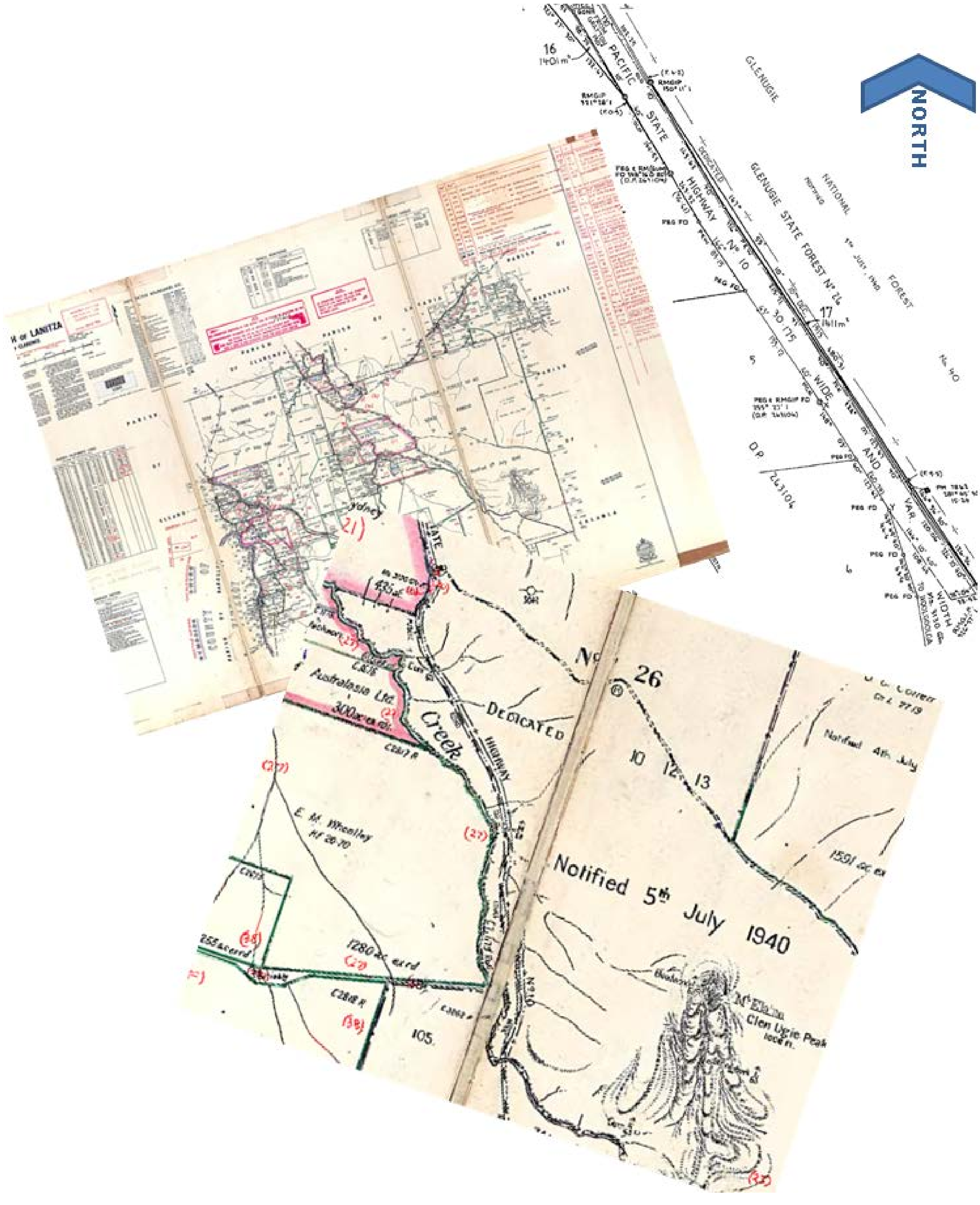

Figure 6: Excerpts from DP719173 and Parish Map of Lanitza.

3.3 Ambulatory Nature of Undefined Public Roads

Another consideration with respect to an undefined public road is the ambulatory nature of such a road, i.e. the road may move at an easy pace in response to physical changes. Using the Pacific Highway as an example, it would be expected that over time the highway in general would have undergone a number of physical changes responding to the needs of modern day road transport and safety considerations. Therefore, would continual improvements to the highway over time to accommodate for larger vehicles and traffic of higher speed (e.g. involving widening or changes in alignments) be considered to be simply changes from time to time due to changing physical conditions? On this basis, an undefined public road remains ambulatory in nature up until such time as it is defined and fixed by survey.

3.4 Determination by Indirect Means

The issue of defining the existing highway corridor was further complicated as parts of the existing formation were obliterated by the Glenugie Upgrade works and no physical evidence remained in parts to confirm its location prior to the works. Fortunately, RDM was able to obtain an electronic copy of the detailed ground survey undertaken by RMS for the engineering design of the Glenugie Upgrade. RDM was then able to extract the control traverse from the file. Of particular interest to RDM were connections to currently existing state survey control marks such as PMs and other current marks.

Through this process, RDM was able to extend the survey control to connect to the same marks and thereby undertake an adjustment of the original detailed ground survey to obtain a strong correlation with the survey control for the acquisition survey. It was then necessary to ensure the actual detail picked up pre-construction was of sufficient accuracy to be relied upon to re-establish the location of the Old Pacific Highway. This was achieved by comparing details of the old highway in areas where it was not disturbed as a result of the new road works. Again, the comparisons between the two models provided a strong correlation sufficient to adopt the detail survey for determining the location of the Old Pacific Highway.

4 LOOKOUT ROAD

Referring back to the Government Gazette of July 1940 (Figure 4), the notification also excluded another road: “public road 1 chain wide in a north-westerly direction from the south-western corner of portion 109”. The road, in this case, was Lookout Road which was a minor gravel side-road connecting properties through the State Forest to the Pacific Highway (Figures 7 & 8). The whole of Lookout Road within the corridor had been obliterated as part of the construction works, leaving no physical evidence of its previous location. Again, the survey had to rely on indirect means to establish its location prior to the upgrade, utilising the previously mentioned detailed ground survey.

The definition of Lookout Road was expected to be a relatively straightforward process of simply overlaying the RMS detail survey over the established cadastral model to fix its alignment. However, this was not to be the case. It was not that the detail was incorrect; it was simply that the location of Lookout Road was noticeably different to that shown on the Parish Map of Lanitza. This difference was in the order of 500 m (Figure 9).

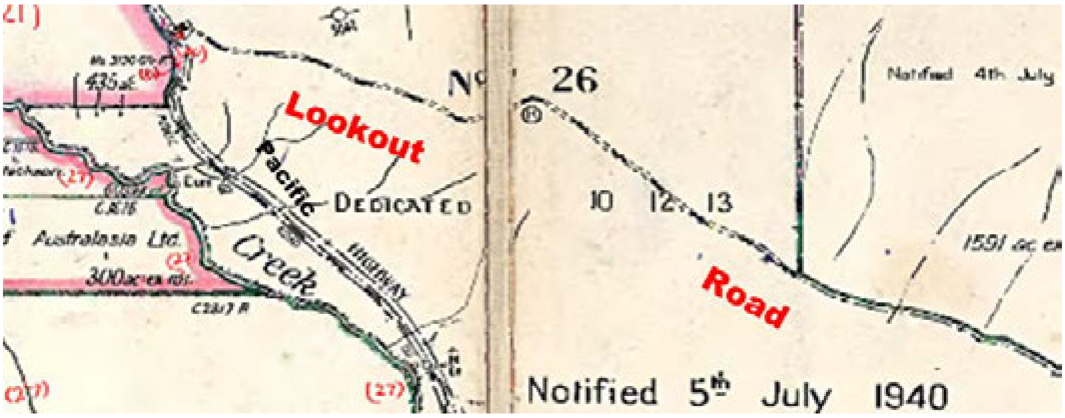

Figure 7: Excerpt of Parish Map showing Lookout Road.

Figure 8: A typical view of Lookout Road outside of the corridor.

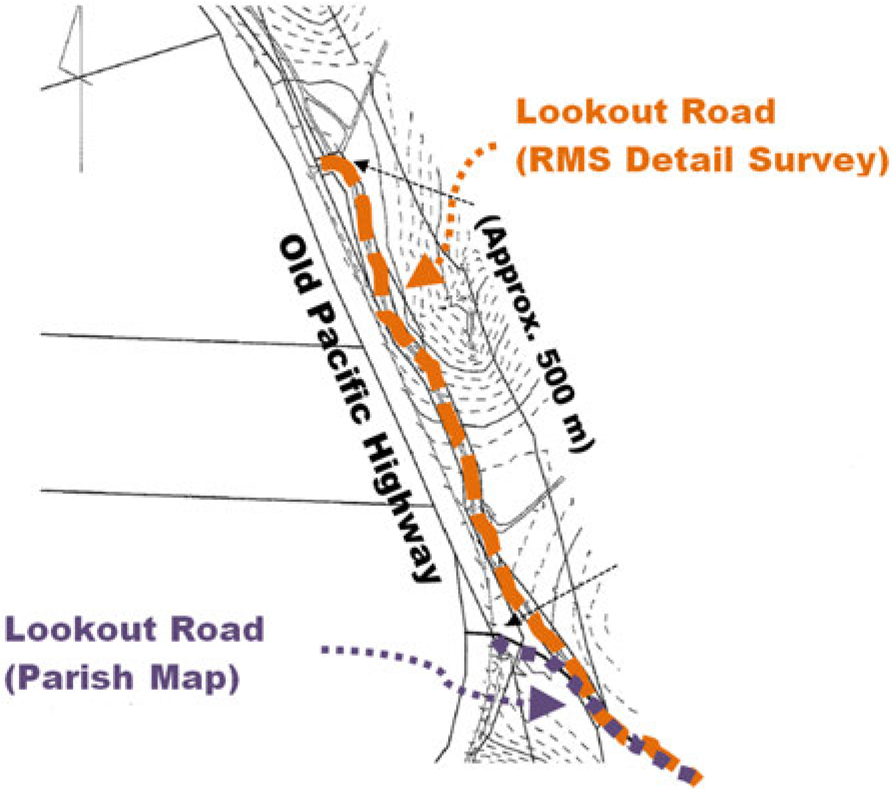

Figure 9: Lookout Road showing variation between RMS detail survey and Parish Map.

As previously determined, an undefined public road is considered to be ambulatory in nature. However, to what extent should this principle apply? Does the notion of ambulatory extend to 500 m?

This raised a number of questions with respect to what should be the underlying principles of an undefined public road. As stated earlier, there was scant information in regards to such roads, and accordingly it was necessary to adopt a common sense approach backed up where possible by already established principles for similar survey situations. In this case, the principle applying to 'easement over a track in use' as described in the NSW Registrar General's Directions seemed apt (LPI, 2015): "the easement site might vary, from time to time, due to changing physical conditions". This concept is well understood, but what if the track (or in this case road) varies by 500 m? Is it the same track or road?

The relocation of the Lookout Road intersection appears to have been a response at some point in time to the need for a safer access point to the Pacific Highway. A public road implies that the road is available for the general public to cross and re-cross at all times. If access to the Pacific Highway was denied temporarily whilst some maintenance work was being undertaken or that the change simply provided an alternate/preferred access to the highway, it would generally be accepted that the road would remain in its original location.

But what if access was fenced off to deny any further access? Would it be deemed that the road had simply varied due to changing physical circumstances? Coming back to the easement over track in use analogy, if a track in use moves significantly to avoid a land slip, fallen tree, or for some other reason, does the easement over the track in use move? RDM took the view that the easement over a track in use would move unless some timely action was taken to restore the track to its near original location.

Before accepting that Lookout Road had physically moved to its new location, based simply on the earlier detail survey, it was necessary to understand whether access to the highway still existed prior to the construction works. In this regard, with the help of RMS, RDM was able to obtain historical images from RMS's Global-Inertial Positioning Systems Image Capture for Asset Mapping (GIPSICAM) system dated February 2007 (Figure 10). The GIPSICAM system is set up to photographically record geo-referenced images along the Pacific Highway at 10-metre intervals on a 2-year rotation.

Accordingly, RDM was able to view the area of interest and confirm that no physical road connection existed between Lookout Road and the Pacific Highway at or near the original location. The images showed Lookout Road running near parallel to the highway and with access denied by an old post and wire fence, which suggested the access had been closed off for some considerable time. Consequently, Lookout Road as an undefined public road was deemed to have moved with its formation to its new location some 500 m north from where shown on the Parish Map.

Figure 10: Images from GIPSICAM (2007) showing Lookout Road.

5 CONCLUDING REMARKS

The acquisition survey for the Glenugie Upgrade raised some unusual issues relating to ‘undefined’ public roads, which required a common sense approach to resolve them. The notion of ambulatory suggests variations over time in response to changing circumstances. However, the notion is simply moving at an easy pace, it does not imply sudden movement.

In the case of sudden movement, the analogy between an ‘easement over a track in use’ and an undefined public road appears to be more suited, given the circumstances where a track (or, in this case, an undefined public road) varies from time to time due to changing physical circumstances. The description of an ‘easement over a track in use’ does not imply whether

the variation needs to be sudden or gradual, nor does it imply that the change needs to be minor or major in extent. Presumably, it could be either or both.

However, the idea of the road moving some 500 m still required considerable thought and appropriate justification that the road was, in fact, still the same road. The test was whether the change was temporary or permanent in nature. The evidence provided by the RMS GIPSICAM system provided sufficient proof that the road had, in fact, changed and the change was permanent in nature. The GIPSICAM photographs indicated that access to the Old Pacific Highway from Lookout Road at or near the original location had been denied for considerable time by the fencing apparent at the time.

The survey and resulting investigations established a number of principles as suitable in the future definition of an undefined public road:

1. The road corridor extends centrally over the road's formation.
2. The road formation may vary from time to time due to changing physical conditions.

The resultant survey was accepted by RMS and subsequently registered by Land and Property Information (LPI) as DP1193563 on 3 March 2014.

ACKNOWLEDGEMENTS

Dealing with such an unusual situation obviously required numerous discussions with peers, and in this instance the following individuals are gratefully acknowledged:

- David Anderson, Registered Surveyor and Engineering Services Manager, Northern Region, NSW Roads and Maritime Services – for his assistance and support in discussing the issues and obtaining various archival information such as the original detail survey and GIPSICAM imagery.
- Matthew Cooper, Registered Surveyor, Resource Design & Management – for always being available as a sounding board, to bounce ideas off and to offer a considered response.
- Christopher Simpson, Candidate Surveyor, Resource Design & Management – for his diligence and enthusiasm in undertaking the field survey aspects of the project and for his keen interest in the application of common sense to resolve a cadastral issue.

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LPI (2015) Registrar General's Directions: Easement over a track in use, http://rgdirections.lpi.nsw.gov.au/deposited_plans/easements_restrictions/easement_over_track_in_use (accessed Jan 2015).

Managing Big Data in a Small Office: How Can We Best Utilise LiDAR and Aerial Photography?

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ABSTRACT

For the survey industry, 'big data' is exemplified through LiDAR (Light Detection and Ranging) data. This could have been captured via terrestrial or airborne laser scanning (or both combined). Each project can constitute hundreds of thousands or even millions of 3-dimensional points, creating files that are hundreds of megabytes to gigabytes of data. This, in conjunction with aerial photography, combines into 'big (survey) data'. Land and Property Information (LPI) maintains a large database of LiDAR and aerial photography data, negating the need to capture the data in the first instant and being available at a very reasonable cost. This paper explores how surveyors can efficiently and economically lever useful information from this library of data and what usable deliverables this data can provide, apart from pretty pictures and contours. For the surveyor, the first hurdle to using the data is the fact that the majority of survey software packages cannot handle the scale of data involved, or if the software can process the data, the processing power, time and memory required is prohibitive to many of the smaller survey offices. The answer lies in embracing Geographic Information System (GIS) software, along with a rethink of how to utilise the suite of existing survey software packages. GIS software allows us to manipulate big data in a much more efficient manner, integrate other GIS mapping (e.g. vegetation mapping and heritage sites) and opens up the possibility of extracting data for a variety of applications. Using this approach, site investigation, feasibility studies, infrastructure corridor assessment, slope analysis, flood risks, and many other aspects of land development and administration can be managed in a desktop environment efficiently and economically, allowing the targeted application of survey resources to either ground-truth or increase the resolution of the LiDAR data to progress a project to detailed design.

KEYWORDS: *LiDAR, aerial photography, GIS, big data, feasibility.*

1 INTRODUCTION

Light Detection and Ranging (LiDAR) and aerial photography (in this paper referred to as imaging or imagery) in combination can provide a powerful tool for both the public and private sectors involved in the management and development of the built and natural environments. Until recently, this information has required costly investment in data capture, infrastructure and processing of this powerful data. For example, in 2008 the cost of capturing aerial LiDAR data and imagery for 50 ha cost upwards of $70,000 in a non-metropolitan area whereby multi-stage mobilisation of the survey aircraft was required to reach the survey area. This cost did not allow for the investment in more powerful computers and memory, along with the high-end Geographic Information System (GIS) software required to be able to manage the size and complexity of the deliverables from the LiDAR and imagery or the

additional training time of staff required to be able to get the final deliverables from the GIS software. This has, to a large extent, limited the utilisation of this data to large organisations and large projects where the funding, skills and infrastructure are in place to economically and efficiently utilise the data.

Land and Property Information (LPI), a division of the Office of Finance and Services within the NSW government, has been collating digital imagery and aerial surveys to define topography using LiDAR and other technologies over significant areas of NSW since 2008. This data is available for purchase by registered LPI users within both the public and private sectors (with certain licencing requirements). This, in conjunction with the range of ‘simplified’ GIS software packages and the dramatic reduction in the cost of data storage, has brought the substantial deliverables of this data within reach of small survey practices and smaller projects.

This paper explores the process of establishing the nature and quality of the data, the range of deliverables, limitations and integration of the deliverables with other survey software and archiving and backing up of the data for future reference through example data and project case studies.

2 LPI LiDAR AND IMAGERY DATA

2.1 LiDAR Data and Specifications

LiDAR survey data is produced in a spatially accurate point cloud and a 1 m resolution bare earth Digital Elevation Model (DEM). All elevation data is processed to achieve ‘Category 1’ DEM products as described by the ICSM Guidelines for Digital Elevation Data (ICSM, 2008), which specifies accuracies not exceeding 30 cm RMSE (2 sigma or 95% confidence). The general specifications of LPI LiDAR data are listed in Table 1.

It is important to understand what datum the LiDAR data was captured in as this will often not be automatically imported into the GIS software and will need to be manually entered in some cases. This is vital when integrating the LiDAR data with other third party GIS data (as discussed later in this paper).

Table 1: General specifications of LPI LiDAR data.

Horizontal Datum	GDA94
Vertical Datum	(Orthometric) AHD71
Vertical Datum	(Ellipsoidal) GDA94
Projection	MGA Zones 54-57
Geoid	AUSGeoid09
Point Density	Minimum 1 point per metre at nadir (lowest return point of laser scan)

2.2 LiDAR Accuracy

In assessing the suitability of the LiDAR data for a desired purpose, the quality of the LiDAR data must be assessed. In most cases, the accuracy obtained for LPI will be *lower* than that obtained via a commissioned project-specific LiDAR data capture process, but is still of a useable order for feasibility, early design and assessments. Fitness for purpose will be addressed throughout this paper.

The applied determination of accuracy methodology, stated below, is taken from LPI's LiDAR Product Specifications (LPI, 2013). Table 2 lists the ICSM accuracy requirements.

"Vertical accuracy is assessed by comparing LiDAR point returns against survey check points in bare open ground. It is calculated at the 95% confidence level as a function of vertical MSE (as per ICSM Guidelines for Digital Elevation Data, 2008). This is undertaken after the standard relative and absolute adjustment of the point cloud has taken place (i.e. flight line matching and shift/transformation to local AHD). A mean value will also be reported for accuracy which is expected to be close to zero.

Horizontal accuracy is checked by comparing the LiDAR intensity data viewed as a 'TIN' surface against surveyed ground features such as existing photo point targets. To date our analysis of ground comparisons shows that although the vertical accuracy achieved on bare open ground is well within the requirements for Category 1 DEM products as specified in the ICSM Guidelines for Digital Elevation Data, local geoid and height control anomalies may degrade the accuracy on large coastal projects."

Table 2: ICSM accuracy requirements (Category 1) from ICSM (2008).

Vertical accuracy	±30 cm at 95% confidence (1.96 x RMSE)
Horizontal accuracy	±80 cm at 95% confidence (1.73 x RMSE)

2.3 LiDAR Deliverables and Formats

LPI provides its LiDAR data in multiple formats that can provide for a specific end-use (Table 3).

Table 3: Deliverables of LPI LiDAR data.

Product		**File Format**	**Description**
1	Classified point cloud	LAS 1.2	Attributed in accordance with designated classification level (2 km x 2 km tiles)
2	Model key points (MKP)	LAS 1.2	'Thinned' ground points (2 km x 2 km tiles)
3	DEM (1m, 5m or 10m)	ESRI ASCII Grid	'Bare earth' (artefact free) DEM (2 km x 2 km tiles)
4	Intensity image	Compressed ECW	0.5 m resolution laser intensity image (single file mosaic – tiles combined) – only available when entire project area is purchased

2.3.1 Classified Point Cloud

Classified point clouds refer to the class membership of a LiDAR point return. All points begin as 'default', i.e. have no classification and are then allocated a meaningful value (i.e. ground, vegetation, building, etc.) by either automated or manual methods or a mix of both. The classification of each point is held in the LAS file and is read into the GIS software, which can interpret this through colourisation (Figure 1). This can be used as a visualisation tool when no aerial imagery is available to aid in the interpretation of the point cloud.

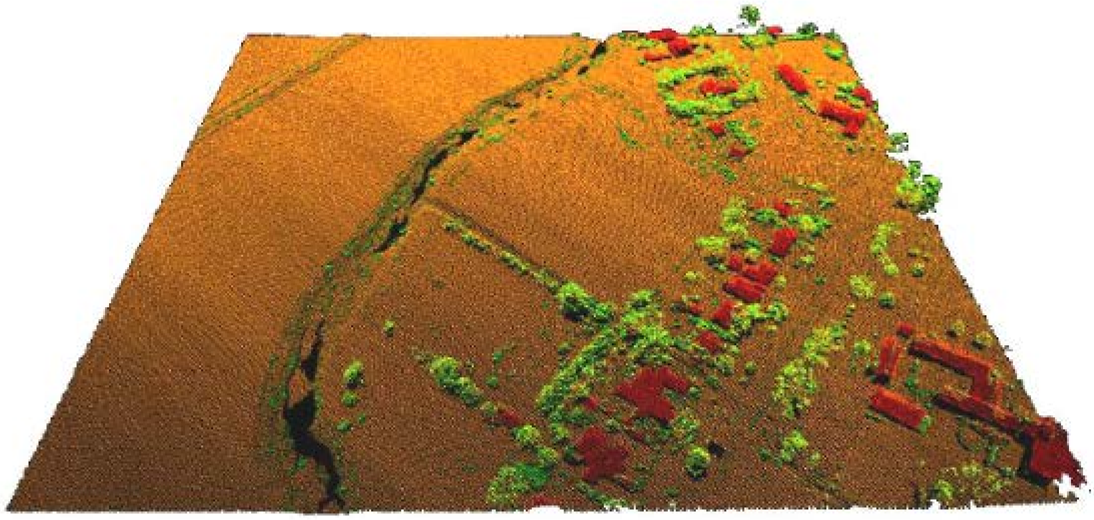

Figure 1: Coloured classified point cloud example (LPI, 2013).

2.3.2 Model Key Points (MKP)

Model Key Points (MKP) are a generalised subset of the original mass points and represent the minimum number of points required to determine the shape of the ground. Experience has indicated that this represents the optimal dataset and should be used in preference to 5 m, or 10 m grid interval point grids as it better represents the surface and is of a considerably smaller file size than the 1 m grid dataset (Figure 2). It should be noted that the minimal point interval is maintained at 1 point per 10 m^2 as can be seen over the paddock areas.

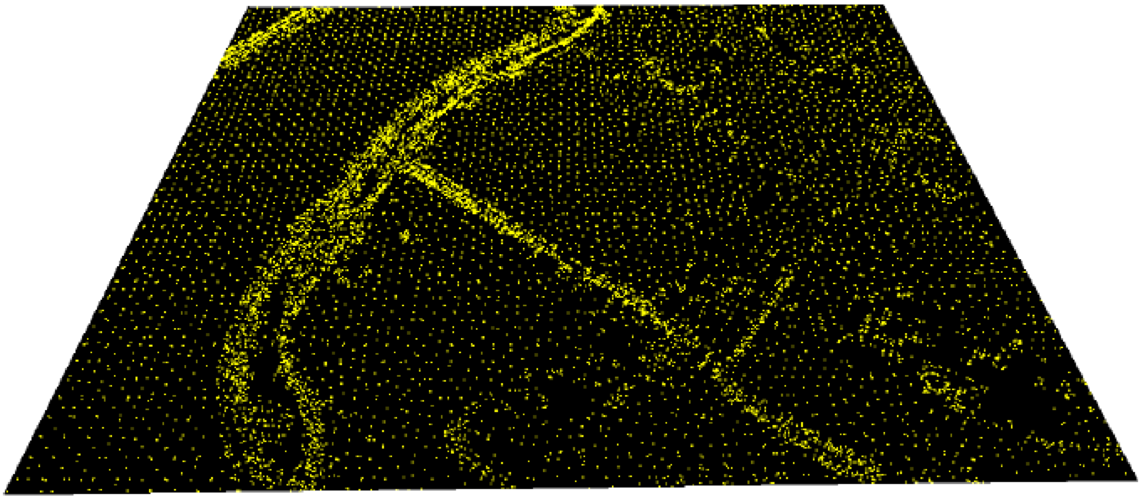

Figure 2: Model key points example (LPI, 2013).

2.3.3 Digital Elevation Model (DEM)

The DEM specifies elevations of the terrain (bare earth z-values) void of vegetation and manmade features. It may incorporate a range of data models such as mass point, TIN, grid or contours and may also include break lines to better represent discontinuous features, thereby improving the overall quality of the DEM (Figure 3).

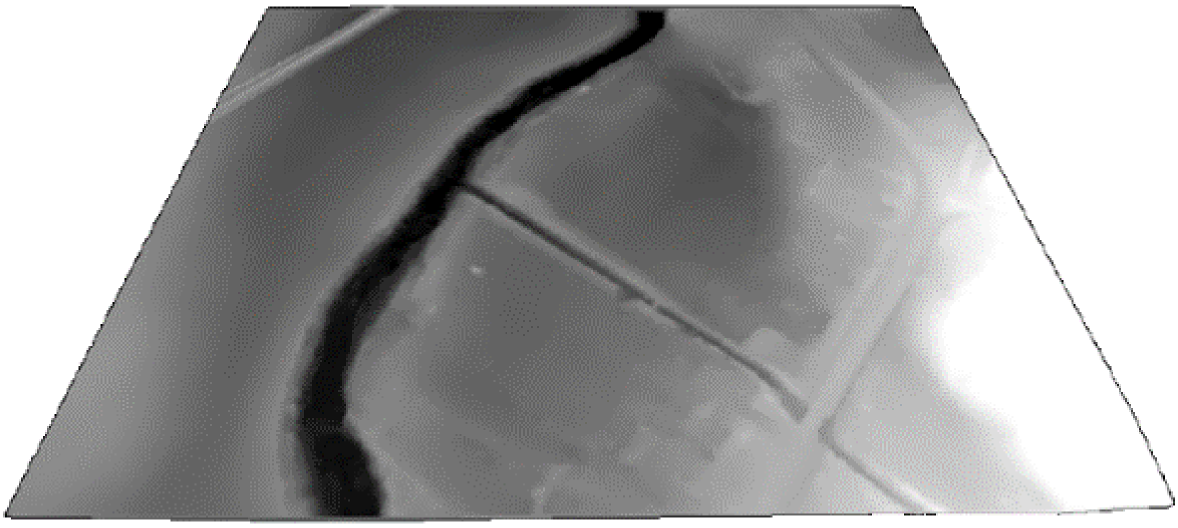

Figure 3: DEM example – shading by elevation (LPI, 2013).

2.3.4 Intensity Image

The intensity image is based on the intensity of the laser pulse return from the LiDAR instrument. This relates to the brightness of the object struck by the infrared laser pulse. Dark grey represents high reflectiveness (e.g. roads and buildings), lighter grey represents low reflectiveness (e.g. grass or other high infrared absorbent surfaces) and white represents nil reflectance (Figure 4). This is a useful tool when no aerial imagery is available. However, it should be noted that these images should *not* be interpreted as black and white images as this would provide false indication of the true contrasts.

Figure 4: Example of intensity image (LPI, 2013).

2.3.5 LiDAR Deliverable Data Sizes

This is where the 'big data' label comes from! Table 4 indicates the data sizes for the Bathurst project area – this has been nominated as it includes urban areas, agricultural areas, hills and plains and thus represents an average of the data sizes that can be expected. This clearly illustrates the large differences in dataset size depending on which dataset is chosen for a particular purpose. This also supports the use of the MKP format where possible as the optimal dataset for size and surface representation.

Table 4: Deliverable nominal file sizes of LPI LiDAR data.

Deliverable	**Nominal file size (averages)**		
	Bathurst		
	Data Size Single Tile	**Single Tile**	**Multiple (20)**
Classified Point Cloud	± 5 million	220 Mb	4.4 Gb
Model Key Points	± 250 thousand	4 Mb	80 Mb
Ground DEM 1 m Grid	± 4 million	35 Mb	700 Mb
Ground DEM 5 m Grid	± 160 thousand	2 Mb	40 Mb
Ground DEM 10 m Grid	± 40 thousand	500 Kb	10 Mb
Total		261.5 Mb	5.23 Gb

It should be clearly understood that the data size can vary considerably from this average and is relative to the nature of the terrain in the project area. The greater the undulation and number of structures, the greater the file size. Conversely, flat areas with minimal structures will have significantly smaller datasets. This will be most evident in the Model Key Points format.

These data sizes also preclude the suitability of the majority of dedicated survey packages in manipulating this data. From personal experience, CivilCAD would take over 5 minutes to load 45,000 points, whereas the GIS software takes only 4 seconds.

2.3.6 Currency of LiDAR Data

The LiDAR data currently available ranges in date from 2009 through to 2014. It is important that the currency of the LiDAR data be taken into consideration with regard to the end use of the data. In areas with recent significant land development or changes in the natural environment through natural disaster or erosion, the available data may not be appropriate to use and provide false representation of the land use and topography.

2.4 LPI LiDAR Coverage

The LiDAR data coverage does not cover the entirety of NSW. LPI has used a targeted approach whereby the majority of the coastal fringe and most significant towns and their surrounding areas have been included to date. Figure 5 indicates the current coverage as of January 2015 via KMZ file supplied by LPI. This file is available upon request from LPI and allows interrogation of each project area for currency of data and data available. Figure 6 provides an example of the metadata for each project area.

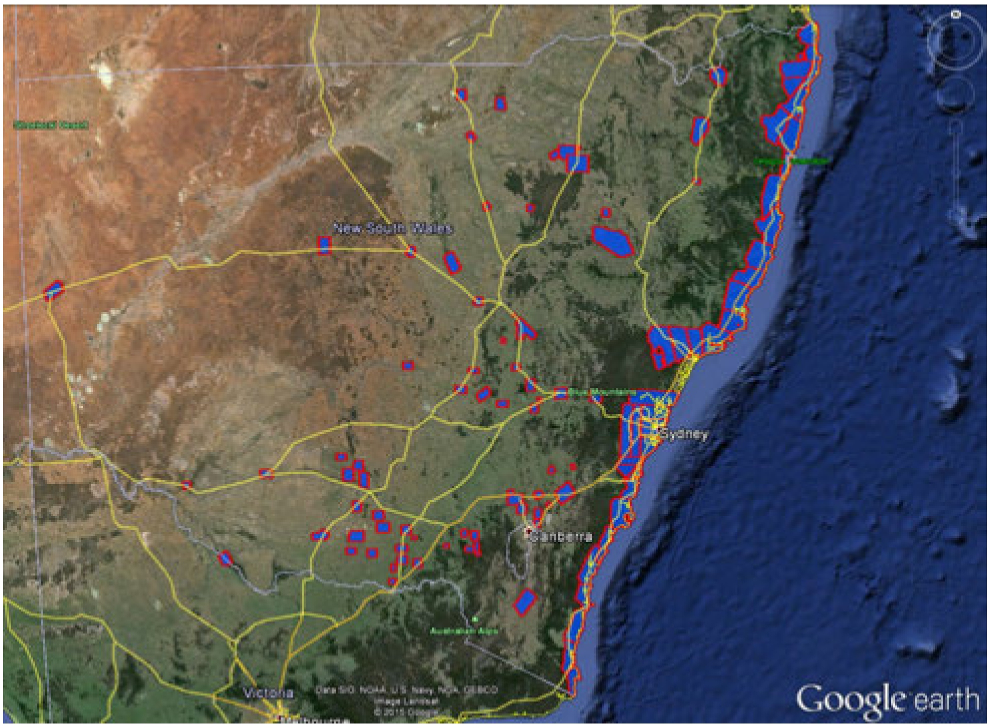

Figure 5: Areas provided with LPI LiDAR data (current as of January 2015).

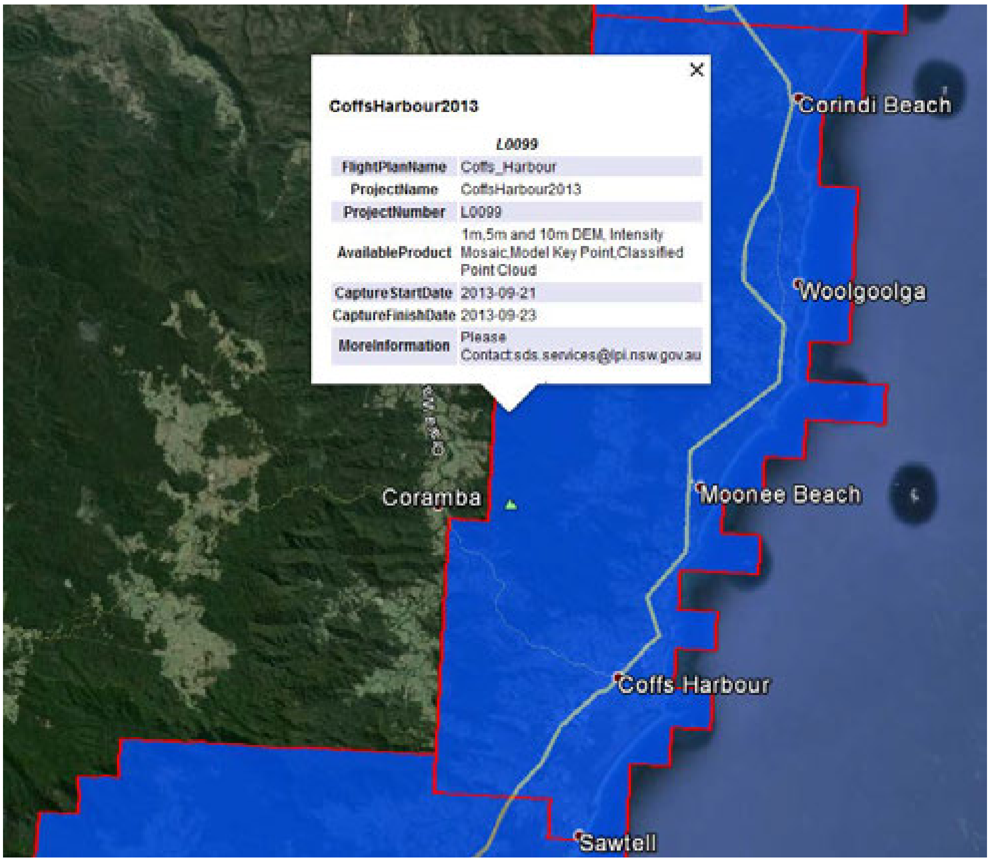

Figure 6: Example of project area LiDAR metadata.

The LPI website (LPI, 2015a) also provides monthly updates via downloadable PDF maps (Figure 7).

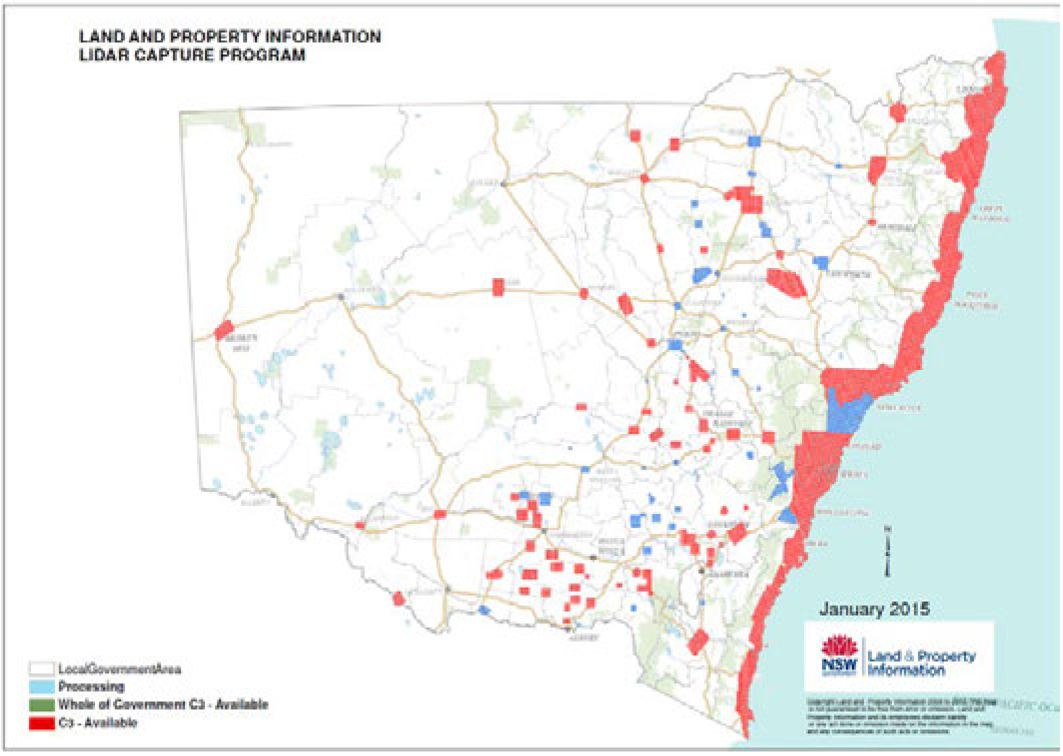

Figure 7: LPI LiDAR capture program as of January 2015 (LPI, 2015).

2.5 Aerial Imagery

2.5.1 What is Available?

LPI's orthorectified aerial photography covers a considerable area of NSW with only some western areas without coverage (Figure 8). This KMZ dataset is provided in the same context as for the LiDAR data and is also imbedded with area-specific imagery data (Figure 9).

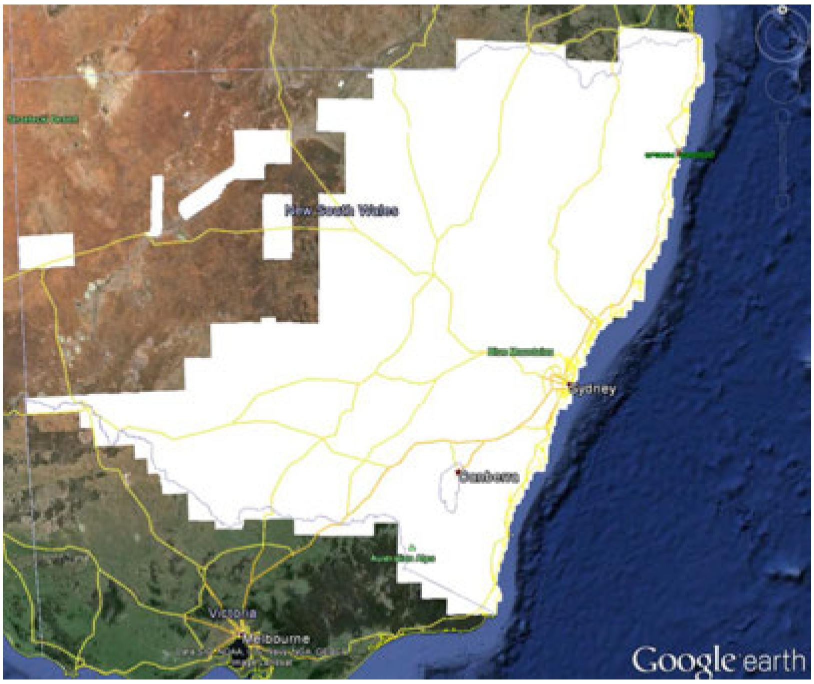

Figure 8: Areas provided with LPI aerial imaging data (current as of January 2015).

Figure 9: Example of project area aerial imaging data (current as of January 2015).

2.5.2 Imagery Data Specifications

As for LiDAR, it is important to understand the datum the imagery was captured in, as this will often not be automatically imported into the GIS software and will need to be manually entered. This is vital when integrating the imagery with LiDAR data and other third party GIS data. Table 5 outlines the general specifications of LPI aerial imagery data.

Table 5: General specification of LPI aerial imagery data.

Horizontal Datum	GDA94
Vertical Datum	(Orthometric) AHD71
Vertical Datum	(Ellipsoidal) GDA94
Projection	MGA Zones 54-57
Geoid	AUSGeoid09
Survey Control	The primary ground control surveys are referenced to AHD as specified above comprising state survey control marks with 'established' GDA94 coordinates and/or 'accurate AHD' heights as defined in the Surveying and Spatial Information Regulation 2012 (NSW Legislation, 2014).

For further information, the reader is referred to the NSW LPI Orthorectified Aerial Imagery and Derived Product Capture and Processing Specifications for Standard Program Acquisitions document (LPI, 2015b).

2.5.3 Resolution of Imagery

The resolution of the imagery is based on the ground sampling distance. This represents the distance between pixel centres measured on the ground and is referred to as the GSD. Table 6 indicates the resolutions for the differing image products.

Table 6: Image resolutions by product type.

Resolution	**Product Type**
10 cm – 20 cm GSD	Town coverage
50 cm GSD	Standard 1:100,000 mosaics
10 cm – 50 cm GSD	Special projects (e.g. emergency services)

2.5.4 Spectrums Available

Various light spectrums are available (Table 7), but not all spectrums are available on every image. Investigation with LPI will determine the spectrums available for the subject area. Although RGB is the most commonly used format due to its visible light spectrum banding, the infra-red images are useful for vegetation mapping. Vegetation absorbs infra-red light differently depending upon the species and health of the plant. This is highlighted in the infra-red spectrum images allowing for a more detailed vegetation analysis.

Table 7: Image colour spectrums available.

RGB	Red Blue Green
RBGn	Red Blue Green and Near Infra-Red
CIR	Coloured Infra-Red
NIR	Near Infra-Red

2.5.5 Areas Available

Table 8 lists the image areas available. It should be noted that 1:100,000 mosaics cover approximately 50 km by 55 km. This does vary from area to area as does the level of overlap between mosaics.

Table 8: Image areas available.

Image Area	Resolution
Whole of town	10-20 cm GSD
1:100,000 map area equivalent (mosaic)	50 cm GSD
Nominated areas (10 km x 10 km) cropped from 1:100,000 mosaic images	50 cm GSD

2.5.6 Image Data Sizes and Formats

Early image capture tends to be in JPG2000 format with later images in ECW (Enhanced Compression Wavelet) format. Approximate sizes for each format for 1:100,000 mosaics are indicated in Table 9. Data size for whole of town and special (emergency) coverage varies due to the size of the capture and can amount to between 500 Mb and 2 Gb, with a few unique cases in excess of 6 Gb. Town coverage file types vary as per the standard coverage sizes (ECW, JP2).

Table 9: Image format and nominal file sizes.

Format (Spectrum)	Data Size
JPG2000 (NIR)	3.5-7.5 Gb
ECW (CIR)	1.5-5.0 Gb
ECW (RGB)	1.0-5.0 Gb

2.5.7 Image Accuracy

Fundamental spatial accuracy of orthorectified aerial imagery conforms to the ICSM Category 1 standard, i.e. Fundamental Horizontal Accuracy (FHA) is better than or equal to ±2.5 x Ground Sample Distance (GSD) at the 95% confidence interval (1.73 x RMSE). This results in an accuracy of ±1.25 m on 50 cm GSD imaging at the 95% confidence interval (LPI, 2015b).

2.6 Data Delivery and Cost

The different forms of transmittal for data are listed in Table 10.

Table 10: Transmittal formats.

Form of Transmittal	Notes
FTP site	up to 20 Gb – download through internet
DVD	
External HDD	purchased from LPI
External HDD	sent in from user, uploaded and returned to user

The low cost of data is one of the leading reasons to investigate how this data can provide project deliverables. For this example, cost is based on a single LiDAR project area and associated cropped 50 cm GSD image. Table 11 provides a breakdown of a typical data acquisition. It should be noted that the project area can vary considerably and should be checked for coverage via the LPI coverage KMZ file in Google Earth.

Table 11: Typical data acquisition cost.

Product	Cost excl. GST
LiDAR full package for project area	$715
10 km x 10 km 50 cm resolution image	$117
Total	**$832**

3 DATA UTILISATION

Owing to the size of the datasets, it is apparent that most survey software will have significant issues or not be able to import the data at all. Some of the higher-end survey software is now able to manage the point clouds but comes at significant cost. This is where investigation of some of the smaller GIS packages is required. It is possible to use software that only costs $100s rather than the $1,000s usually associated with GIS software. These 'simplified' packages can efficiently handle the large datasets and in most cases provide all of the deliverables indicated in this paper.

It should be understood that these GIS software packages are not detailed hydrological modelling packages, and further analysis will be needed by specialists to determine finite outcomes for detailed design. Care should be taken using the settings associated with each of the outputs as this can dramatically alter the outputs. Bad settings equal bad outputs, equals erroneous feasibility studies, equals expensive mistake!

The data derived from using the methodology described in this paper is best applied to the master planning and feasibility stages of projects to help in determining the focused approach only where it is needed, thus minimising the costs of detailed data capture, processing and analysis. The following sections provide examples of what can be generated from the LPI data using a Model Key Point (MKP) file and a 10 cm resolution image.

3.1 Contours and Terrain Models

Contours and terrain models (triangle files) are the simplest and most common output from LiDAR data (Figure 10). However, it should be noted that due the large number of points utilised to create the contour and terrain model, these surfaces can be of considerable size in themselves. Options are usually available within the GIS software that will 'simplify' the contours to some extent, thus reducing the file sizes, care should be taken with this as excessive simplification will cause the representational accuracy of contours to be degraded.

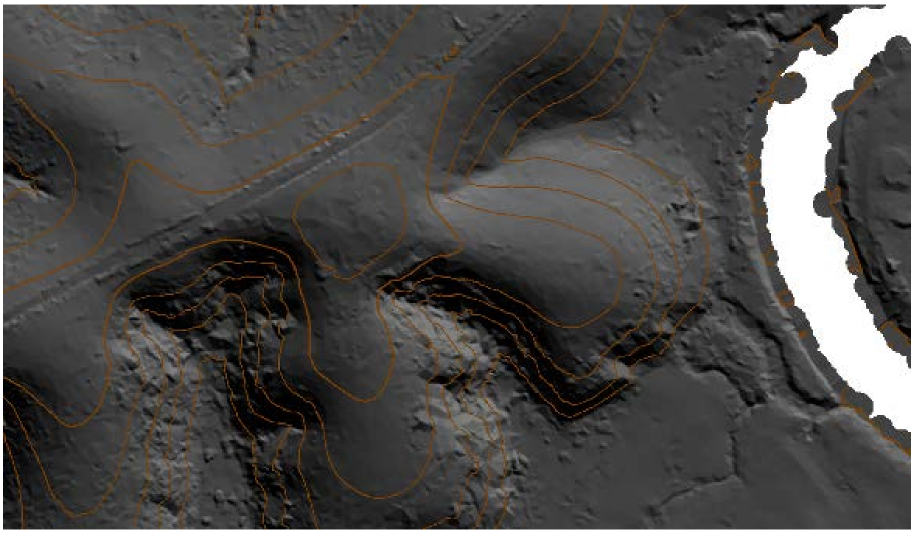

Figure 10: 5 m contour interval over daylight shaded DEM.

3.2 Coloured DEMs

Colourisation of the DEM can provide an insight into the survey area that conventional survey methods would never hope to attain. Definition of natural drainage channels, tracks and roads, erosion areas and a whole variety of natural and built environment features can be teased out of the data. This section provides some examples of what can be achieved. These deliverables are best exported from the GIS software as raster (image) files and inserted into other software in a similar manner to normal background imagery.

3.2.1 Daylight Shading

Daylight shading can be useful in determining what areas will be in sun and what areas will be in shade during the day. Examples of lighting conditions in the early morning and late afternoon are shown in Figures 11 & 12, illustrating that the effect on the rendering can be quite dramatic.

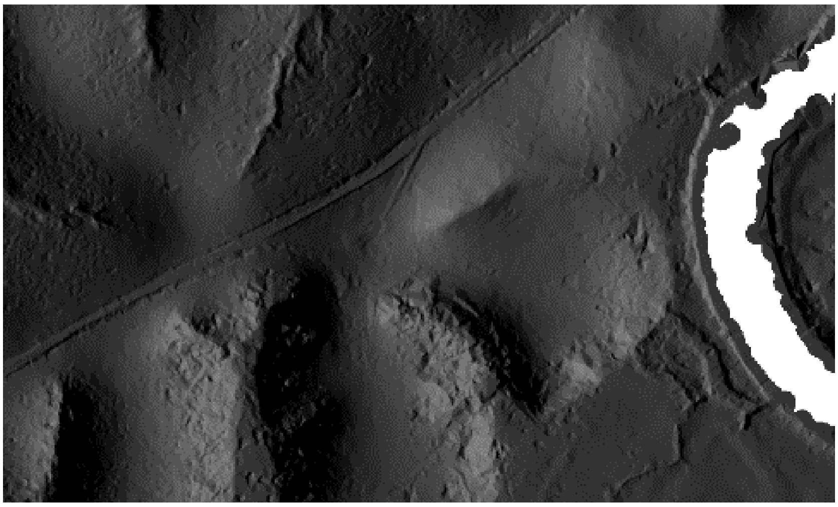

Figure 11: Early morning daylight shading.

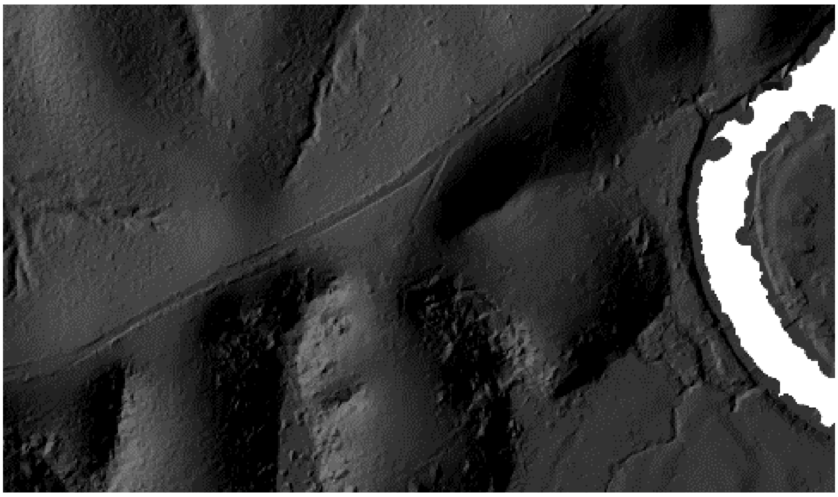

Figure 12: Late afternoon daylight shading.

3.2.2 Colour by Elevation

Colour by elevation can provide easily interpreted relative heights of the DEM. This can be very useful when providing interpretation of drainage over gently sloping areas where contours are less indicative (Figure 13).

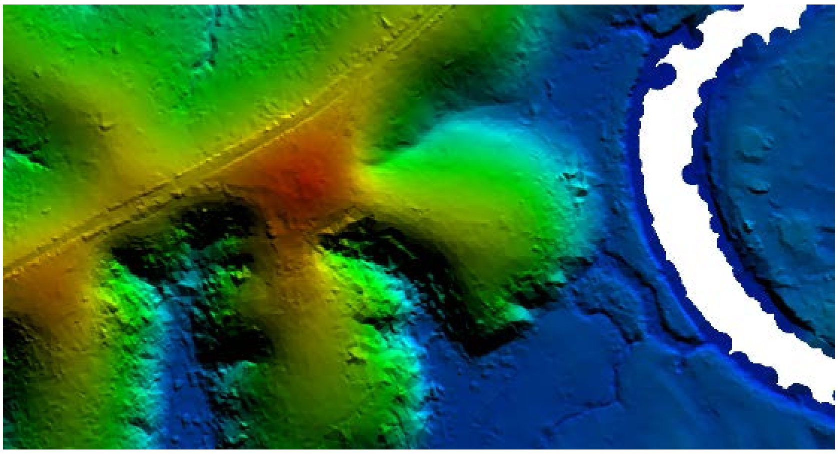

Figure 13: Colour by elevation shading.

3.2.3 Shade by Slope Direction

Shading by slope direction can provide insights into subtle changes in terrain that may not be readily visible in daylight or height by elevation shading. This can be seen in Figures 14 & 15, illustrating 'atlas' shading (general colour representation that you would find on large scale topographical maps) and shading by slope direction, respectively. Historical banks of the waterway become apparent and therefore of assistance in understanding the geomorphology of the subject area. This data is also of interest to farmers and other growers as it indicates areas that are best targeted for planting through identification of desirable aspects.

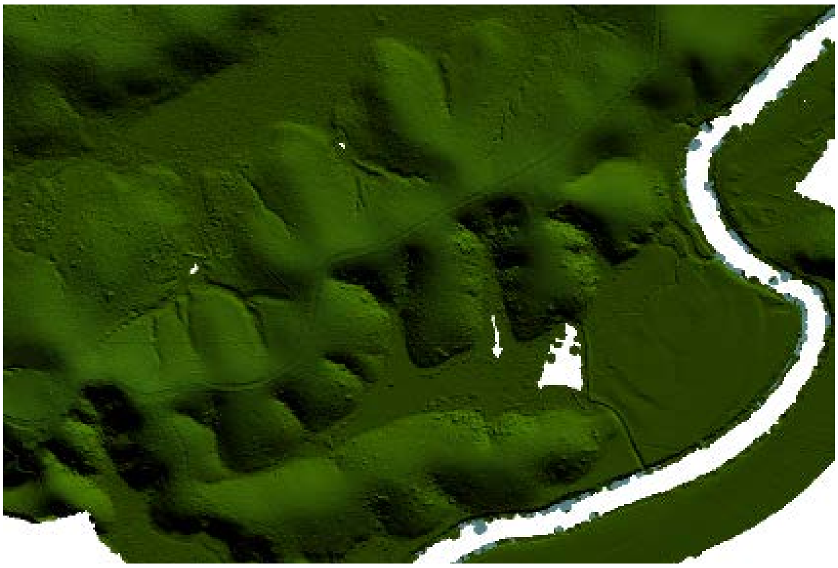

Figure 14: 'Atlas' shading.

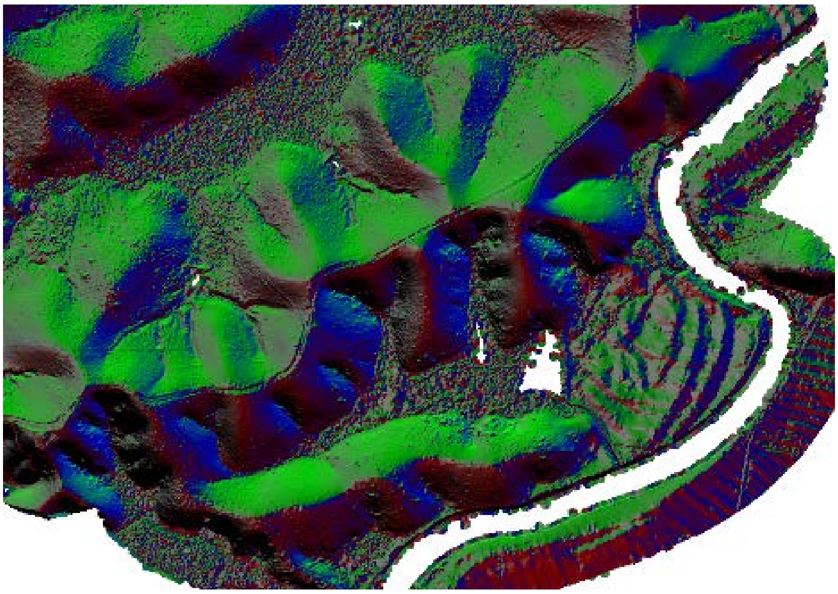

Figure 15: Shade by slope direction.

3.3 Definition of Drainage Channels, Waterways and Catchments

Understanding how an area sheds itself of rainfall is of significant interest to parties involved in all forms of land development and management. Digitisation of published topographic

mapping may not accurately represent the true extent or location of drainage channels and waterways thus limiting the quality of the data for planning purposes. Many of the GIS packages are capable of analysing the surface models and determining the catchment areas, and consequently the alignment of drainage channels and waterways over the project area.

There will often be considerable customisation associated with the level of detail that can be produced. Care must be taken in this area to choose settings that represent waterways in a realistic manner, e.g. telling the software to ignore any drainage channel less than 30 m in length. An example of drainage channel analysis is given in Figure 16, with coloured graphical representation of the catchment areas shown in Figure 17. As previously mentioned, these GIS packages are not capable of full hydrological analysis or categorisation of drainage channels or waterways. This data is indicative and is to be used to determine where further study by hydrological professionals may be required.

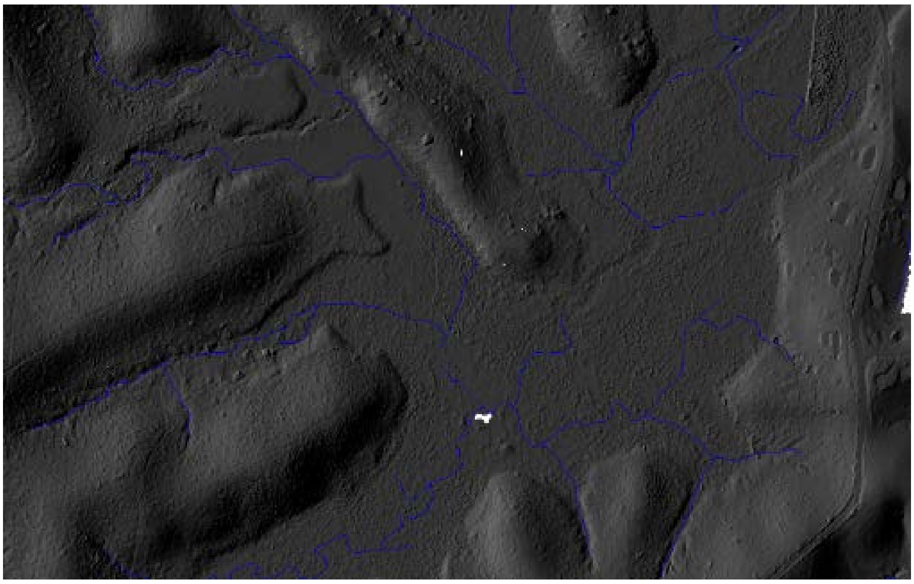

Figure 16: Drainage channels generated from catchment analysis.

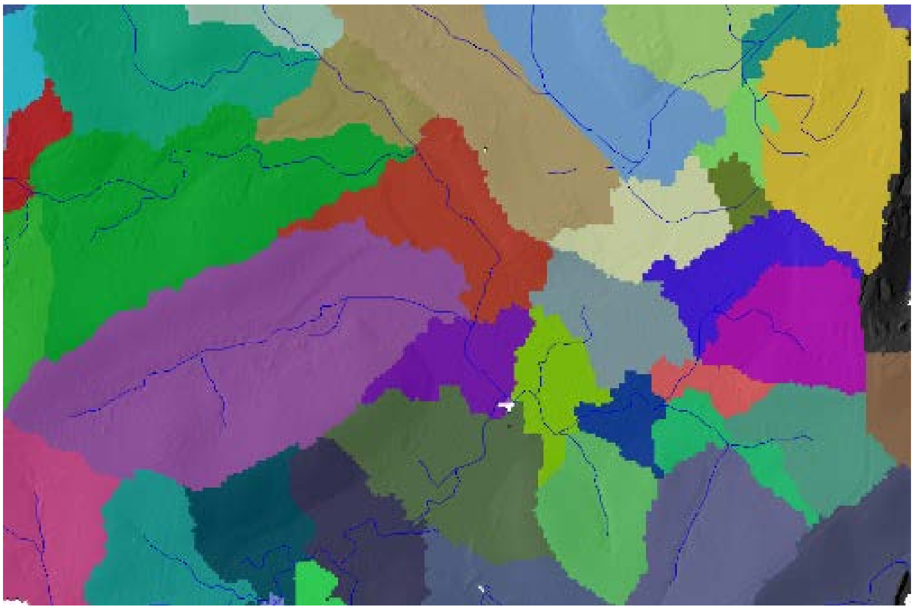

Figure 17: Indication of individual catchments through colourisation.

3.4 Modelling of Flood Scenarios from Rain Events, Storm Surges and Sea Level Rise

Australia is the land of fire and flood. Adding sea level rise, storm surges and in rare cases tsunamis, it is clear that the analysis, understanding and mitigation of risks to life and assets from these events is vital in the future planning of our built environments. Again, the GIS data can assist in assessing these threats. Modelling tools are available that can illustrate sea level rise and impacts of storm surge and flood events (Figure 18). This figure models a 0.25 m water level above a nominated level associated with the waterway. Floodwater inundation is indicated in pink. It is apparent that some buildings were built on pads that mitigate for this and some were not.

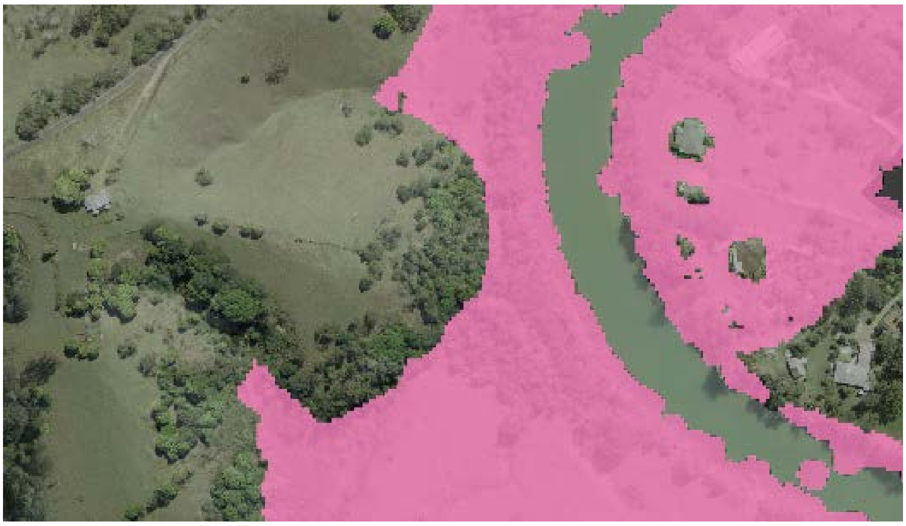

Figure 18: Flood event modelling.

3.5 Slope Analysis

Understanding the range of slope over a project area can be useful for many aspects of assessment and design, e.g. the determination of road alignments to avoid gradients over a particular angle. Figure 19 indicates a colourised assessment of gradient: 0-10º = green, 10-15º = yellow, and over 15º = red.

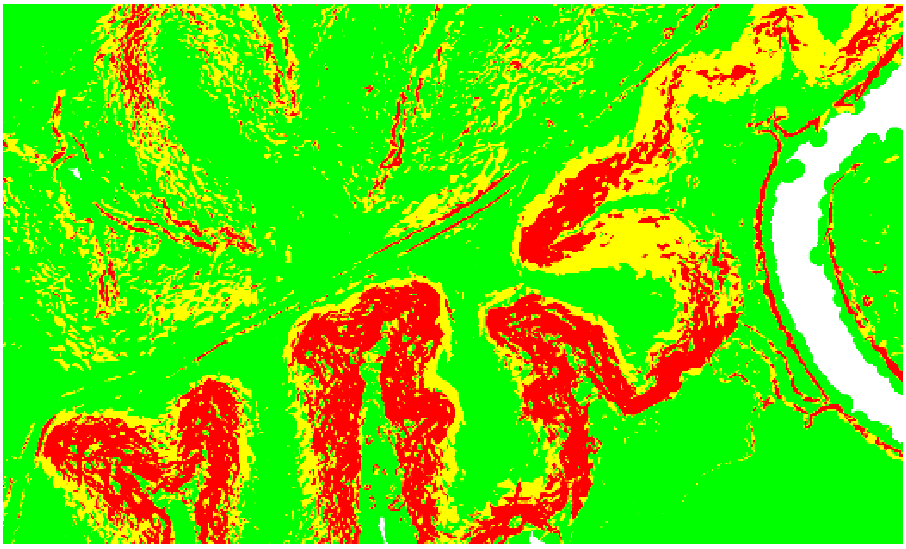

Figure 19: Slope analysis.

3.6 Bushfire Risk Analysis

Continuing with the earlier theme of planning for natural disasters, fire presents as significant or greater a threat than that of flooding. When carrying out a bushfire risk assessment, vegetation and slope are of paramount importance. By combining imagery, slope analysis shaded DEMs and, where possible, LGA detailed vegetation (see section 4 regarding the integration of third party GIS data) analysis, reasonable desktop assessments can be made in establishing Asset Protection Zones (APZ) and determining the level of risk of assets within a project area. Determination of appropriate constrains and criteria should be made by suitably qualified persons in the area of bushfire assessment, given the risk to life and assets bushfires in Australia represent. Figure 20 indicates what can be represented using the above techniques (although LGA vegetation mapping data is not displayed here).

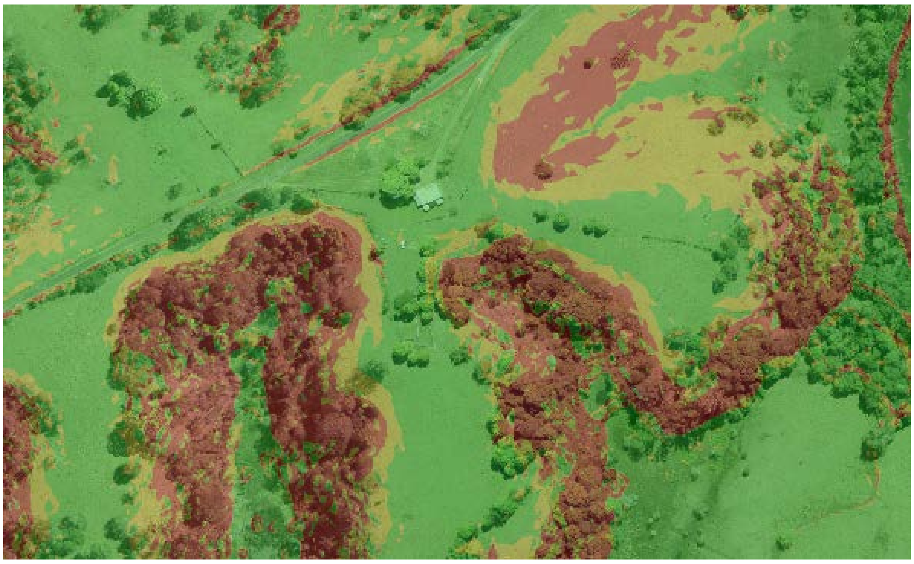

Figure 20: Bushfire risk modelling.

3.7 Visualisation using 'Draped' Images over DEMs

Public consultation associated with land development can be a challenging exercise. Providing realistic imagery of proposals can greatly aid in ensuring the public interpret what is being proposed in an accurate and intuitive manner. Again, the utilisation of GIS data can augment LPI's aerial imagery and proposed designs through integration of DEMs. As an example, Figure 21 shows the original image, while Figure 22 shows the integration with the DEM, introducing natural terrain shading. Figure 23 indicates what can be achieved with 3D perspectives of the existing topography. Finally, Figure 24 introduces the natural constrains of the subject area in terms of slopes, contours, fire and flood.

Figure 21: Standard image.

Figure 22: Opaque standard image draped over DEM to indicate terrain.

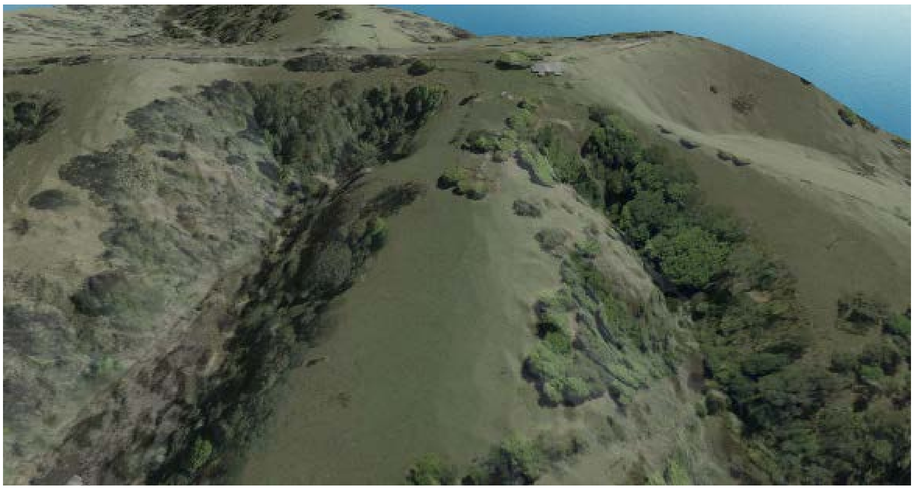

Figure 23: 3D visualisation.

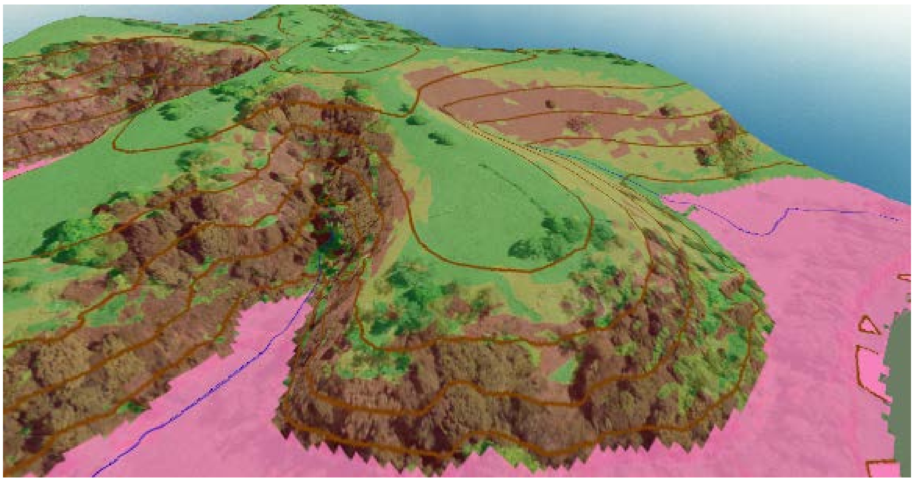

Figure 24: 3D visualisation of a combined constraint plan.

4 DATA INTEGRATION

4.1 Integration of Third Party GIS Data

As GIS software is being utilised to augment LiDAR and image data, it allows for the importation and integration of third party GIS data. For example, most local councils will have a GIS department managing database that can cover all aspects of town planning, council assets, detailed vegetation mapping, contaminated lands, etc. Many councils are prepared to issue this data for a fee thus allowing the integration of high-value data when determining service corridors, assessing feasibility studies and strategic planning that accurately take into consideration local authority constraints. Figure 25 provides an example of typical detailed vegetation, flooding and property boundary data courtesy of Coffs Harbour City Council's online mapping portal (Coffs Harbour City Council, 2015).

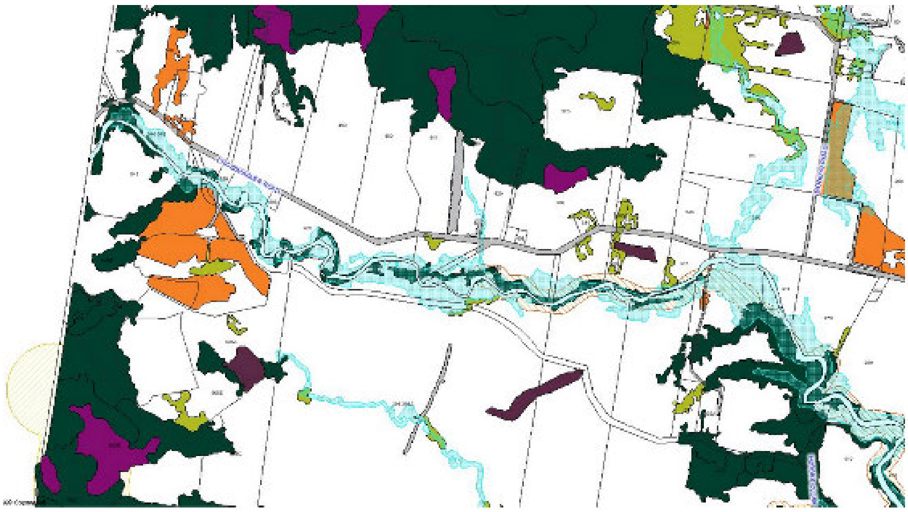

Figure 25: Example of Local Authority GIS Local Environment Plan mapping.

4.2 Integration with Survey or CAD Software

Most of the GIS packages will provide numerous options regarding the export of vector (CAD) and raster (image) data. Experimentation will be needed to determine the best formats

and export settings that provide the optimal resolution, detail and file size to be integrated into the survey software packages or other CAD packages. Once these have been determined, they should be integrated into standard procedures and applied consistently.

4.3 Management of Datasets

With data being exported from GIS datasets into CAD and third party GIS data being imported into LiDAR data along with information from the survey software, it is vital that procedures are developed and enforced that manage the origin and currency of data. This can be achieved in any manner of ways and is best developed by each individual practice to integrate with their current quality assurance and document control systems. If these processes are not adhered to, there is great risk of superseded data being used along with confusion as to where particular data should be held in the GIS database or in the survey software.

5 STORAGE, MANAGEMENT AND BACKUP OF LiDAR AND IMAGE DATA

Consideration should be given as to how to store, access, backup and archive LiDAR and aerial imagery. Each practice will have its own current data storage solution, thus storage and backup solutions of LiDAR and image data will be just as individual. The findings and recommendations stated below are based around a small business server solution that can be found in many small practices. It is recommended that consultation with an IT specialist be undertaken to determine what would be the most suitable solution for individual practices.

5.1 Storage of Data

Where only very intermittent small projects are envisaged, the standard protocols used for conventional survey data are still the best solution. However, if multiple smaller projects are envisaged (or even one or two large projects), standard server storage solutions will struggle, causing the slowing of all data transfer between servers and workstations. If the latter scenario is expected, then it is recommended that dedicated hard drives be allocated to the existing server to specifically manage these large datasets, stored in a similar file structure as the original project drive. This will then avoid the issue of compromising current project drive storage and access speeds. Most small business servers will have capacity for several hard drives, so the cost of implementing this process would be in the order of $400 for each additional 2TB hard drive and around $100 for installation.

5.2 Management of Data

It is recommended that the existing GIS software is utilised to manage the collation of the LiDAR, imaging and any third party GIS data – these can be saved in the same location as the LiDAR and aerial imagery datasets. This will avoid most issues where data gets moved independent of the project database and thus needs rebuilding. The GIS database will usually hold the correct order of data, configuration associated with DEM shading, image transparency settings, etc. This will save considerable time reloading data when revisiting a project in the future.

5.3 Backup and Archiving of Data

As noted in regards to storage, the practice’s standard backup and archiving protocols should cope with a few small projects. However, if the move to independent hard drives is made on

the server, this should also be replicated in the backup solution. In most cases, this can be via external multi-TB hard drives that are rotated into fireproof offsite storage. These drives can be purchased for about $200-300.

5.4 Cloud Storage

At this time cloud storage is not recommended as a storage solution. The sheer size of the datasets preclude this as a solution for small to medium practices. However, with the continued expansion of online storage, data transfer speed and (hopefully) improved encryption and security, this situation may change in the future.

6 CONCLUDING REMARKS

Utilisation of LPI's LiDAR and aerial imagery datasets through GIS is possible for even the smallest of survey practices. Cost of data, equipment and software have drastically reduced in recent years, enabling a broad range of deliverables to be provided at a reasonable cost. However, the data's limitations must not be underestimated. Basing detailed design on these outputs is highly dangerous due to the resolution, accuracy and currency of the LPI data. Where possible, it is recommended that ground truthing of the data be carried out on sample areas over the extent of the project area. This will provide some 'dead reckoning' of the data and lends confidence to stated relative accuracies of the data.

The data's utilisation lies in the early stage feasibility and planning studies, where broad decisions can be made along with targeted detailed surveys of key areas to allow for detailed design and assessments via conventional terrestrial surveys, Global Navigation Satellite System (GNSS) surveys or further LiDAR surveys.

Recent investigations into current costs of aerial LiDAR and image capture have indicated that high-resolution capture (30 cm horizontal, 15 cm vertical, and 5-10 cm resolution imagery) can be captured at reasonable cost, provided capture can be planned far enough ahead. A 50 ha project area within the coastal fringe of NSW would cost in the order of $7,000 excl. GST (January 2015).

This paper provides no recommendations with regard to the integration of terrestrial laser scanning data, solid modelling or particular GIS packages to achieve the examples given. It is for the individual practice to determine the 'best fit' of software and infrastructure to integrate with the practice's budget and current processes.

ACKNOWLEDGEMENTS

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- Tim York, Manager Atlass Aerial Surveys – provision of costing information.
- Coffs Harbour City Council GIS Department.

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The Survey Audit Program

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ABSTRACT

Land and Property Information (LPI) is responsible for administering a range of legislation, which provides the framework for land titling and conveyancing, surveying, valuation and related matters that underpin the economy of New South Wales. In supporting the statutory functions of the Registrar General, Surveyor General and the Valuer General, LPI protects the integrity of the Torrens Register and maintains and promotes standards that ensure secure, consistent, high-quality spatial and valuation information is provided to the community. LPI's core services, provided on behalf of the Registrar General, include examination and registration of survey plans and property dealings, issuing of certificates of title and conversion of Old System land to Torrens title. Each plan lodged for registration at LPI undergoes a desktop examination by plan examiners of the Titling and Registry Services (T&RS) unit to ensure that the plan complies with all relevant acts and regulations. The plan may also be subjected to a field audit, undertaken by surveyors or other officers of LPI's Cadastral Integrity Unit (CIU), to further assess its regulatory compliance. A plan may be audited either before or after its registration, with the majority of audits to be performed on unregistered plans.

The objectives of the audit program are:

- *To establish procedures for conducting audit surveys so as to assess and measure in the field a plan's or registered surveyor's compliance with the requirements of the Surveying and Spatial Information Act and Regulation, Conveyancing Act and Regulation, Registrar General's Directions, Surveyor General's Directions and all other relevant legislation including that pertaining to Community and Strata schemes.*
- *To improve the quality and consistency of Deposited Plans lodged for registration.*
- *To put in place a process for the education of surveyors.*
- *To establish a mechanism for referring cases of complaint and sustained non-compliance to the Board of Surveying and Spatial Information (BOSSI).*

This presentation provides background on survey audits and outlines the procedures followed by LPI during such audits. More information can be found in the Cadastral Integrity Unit Audit Survey Procedures, which are available for download from the LPI website at http://www.lpi.nsw.gov.au/__data/assets/pdf_file/0003/174585/CIU_Audit_Surveys_Procedures.pdf.

KEYWORDS: *Cadastral surveying, survey audit, legislation.*

The Position and Verticality of Structural Steel

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ABSTRACT

The Australian Standard AS 4100 Steel Structures outlines the relevant positional and verticality accuracies for the erection of steel structures. The determination of these accuracies is a common task undertaken by SME (Structural, Mechanical & Engineering) surveyors. Available equipment, time constraints and access to structures often dictate the methodologies adopted. This paper explores various equipment and methodologies for determining verticality and position and their relative pros and cons. Examples are drawn from work on the Nammuldi Below Water Table Project undertaken in the Pilbara region of Western Australia.

KEYWORDS: *Position, verticality, AS 4100.*

1 INTRODUCTION

A common task for SME (Structural, Mechanical & Engineering) surveyors is the determination of position and verticality of steel structures. A typical structure could be the support frame for a train load out (TLO), as shown in Figure 1. This million-dollar structure and associated conveyors sits primarily on half a dozen columns, and the integration between the TLO and the rail line, the conveyor etc. hinge primarily on the position, level and verticality of the TLO.

Figure 1: Train load out structure.

The availability of equipment, site conditions and time constraints all impact on the methodologies adopted by SME surveyors. They also have available to them a number of traditional and modern methodologies, and sometimes the combination and permutations of equipment, methodologies and constraints can be daunting. This paper looks at a number of methodologies used for determining position and/or verticality and then focuses on one particularly methodology that combines all the measurements (position, verticality and level) into one method. It is by no means a definitive solution but one that works well and addresses a number of the constraints mentioned above.

2 AS 4100 STEEL STRUCTURES

The Australian Standard AS 4100 Steel Structures outlines (among other things) the relevant positional and verticality accuracies for the erection of steel structures. The main sections under consideration in this paper are 15.3.2 *Column base* and 15.3.3 *Plumbing of a compression member*, outlined as follows (Standards Australia, 1998):

15.3.2 Column base
15.3.2.1 Position in plan
The position in plan of a steel column base shall not deviate from its correct value by more than 6 mm along either of the principal setting out axes.
15.3.2.2 Level
The level of the underside of a steel base plate shall not deviate from its correct value by more than ±10 mm.

15.3.3 Plumbing of a compression member
The alignment and plumbing of a compression member shall be in accordance with both of the following requirements:
(a) The deviation of any point above the base of the compression member from its correct position shall not exceed height/500 or as follows, whichever is the lesser:
(i) For a point up to 60 m above the base of the member … 25 mm.
(ii) For a point more than 60 m above the base of the member … 25 mm plus 1 mm for every 3 m in excess of 60 m up to a maximum of 50 mm.

3 POSITIONING

Perhaps the least complicated task to be undertaken is the determination of position. Figure 2 shows the cross section of a column with a six-hole plate at its base. If the anchor bolts have been positioned correctly, it would be assumed that the column is in its correct location but variations in construction, welding etc. may mean the actual column may not be in the correct location.

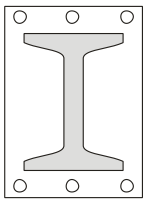


Figure 2: Cross section of a sample column and base plate.

The simplest approach is to measure the width and breadth of the column and mark, the centre points and then calculate the position of these marks relative to the design position. In the example shown in Figure 3, column A has been measured and the centre of the column found to be out of position. The column is 4 mm from the design position along the Y axis and 2 mm out of position along the X axis. The dimensions do not exceed the acceptable tolerance of ±6 mm in either of the primary axes (see section 15.3.2.1 of AS 4100), so therefore the position of the column meets AS 4100 requirements.

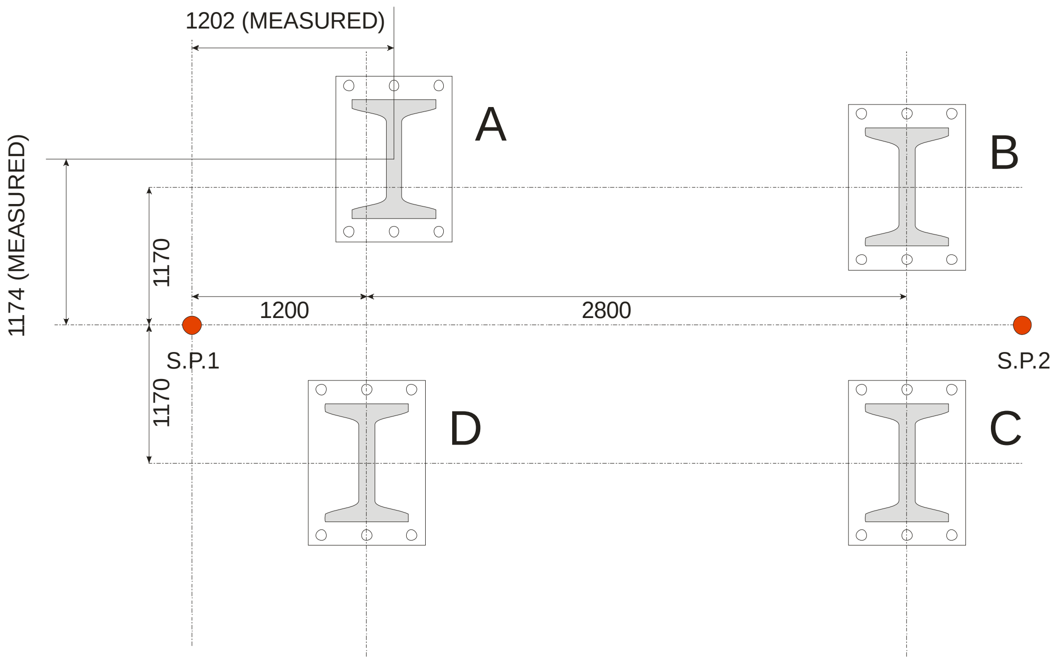


Figure 3: Column measurements.

4 PLUMBING

There are a number of approaches for measuring the verticality of a column, including direct measurement by plumb bob, optical laser plummet and total station. Occam's razor (Encyclopaedia Britannica, 2015) suggests that sometimes the simplest solution is often the best (apologies to William of Ockham c. 1287–1347/49 for the loose interpretation). The verticality of a column can be determined quite simply with a plumbob or spirit level (Figure 4).

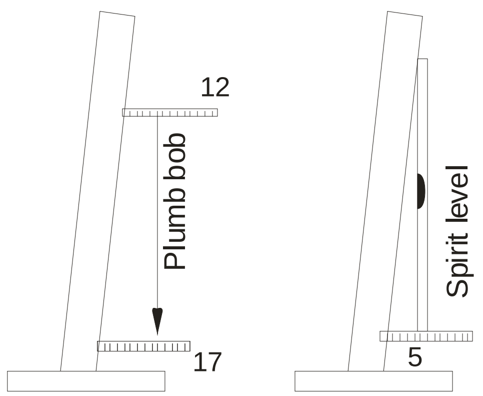


Figure 4: Determining column verticality using plumb bob and spirit level (leaning 5 mm).

An alternative method involves sighting a point near the top of the column and then observing the same point near the base and determining the difference. This requires the line of sight to approximate the axis and hence requires two setups to determine the 'lean' of the column in each axis (Figure 5).

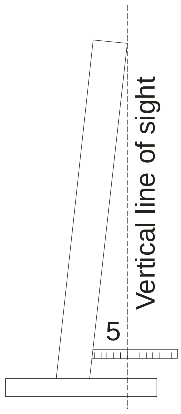


Figure 5: Determining column verticality using a total station (leaning 5 mm).

5 ACCESS AND SITE RESTRICTIONS

One of the most significant limitations to checking location or verticality is physically accessing the columns and base plates. Site safety requirements often dictate a drop or exclusion zone around working areas and this significantly restricts access to columns and base plates. Figure 6 illustrates the exclusion zone that may surround the columns and provides a photo of an exclusion zone (yellow hard barricading) due to the elevated work platform (EWP) being raised into the structure.

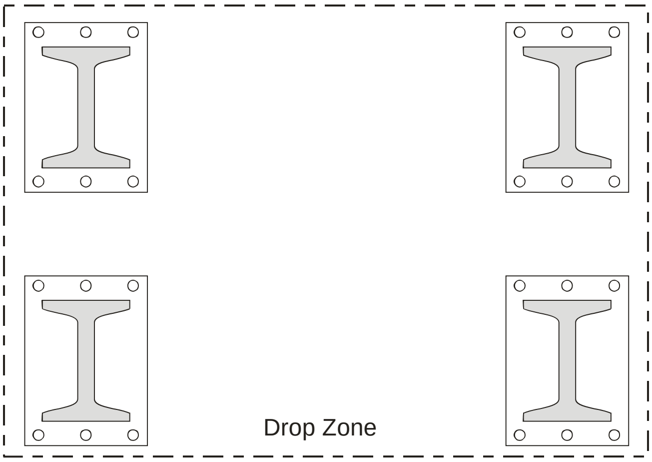


Figure 6: Drop or exclusion zone (left) and drop zone example (right).

6 REMOTE METHODOLOGY

In an ideal world, accessing the columns to measure location and verticality would be easy but in practice site access is an issue and therefore a different, remote methodology is required. It should be noted that the following methodology can be modified for sites where access is fully available or partially limited, but the basic premise is that access is not readily available.

6.1 Assumptions

For the sake of this paper three assumptions are made, based on current work practices.

6.1.1 Assumption 1: Standard Beam/Column Dimensions

The first assumption is that the dimensions for the columns are standard across the project. Although there are slight variations, this assumption has been mostly valid on site and readily been checked prior to construction. The dimensions most relevant for this paper are the distance between the flanges and the web (Figure 7).

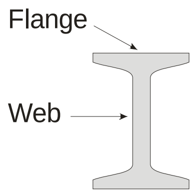
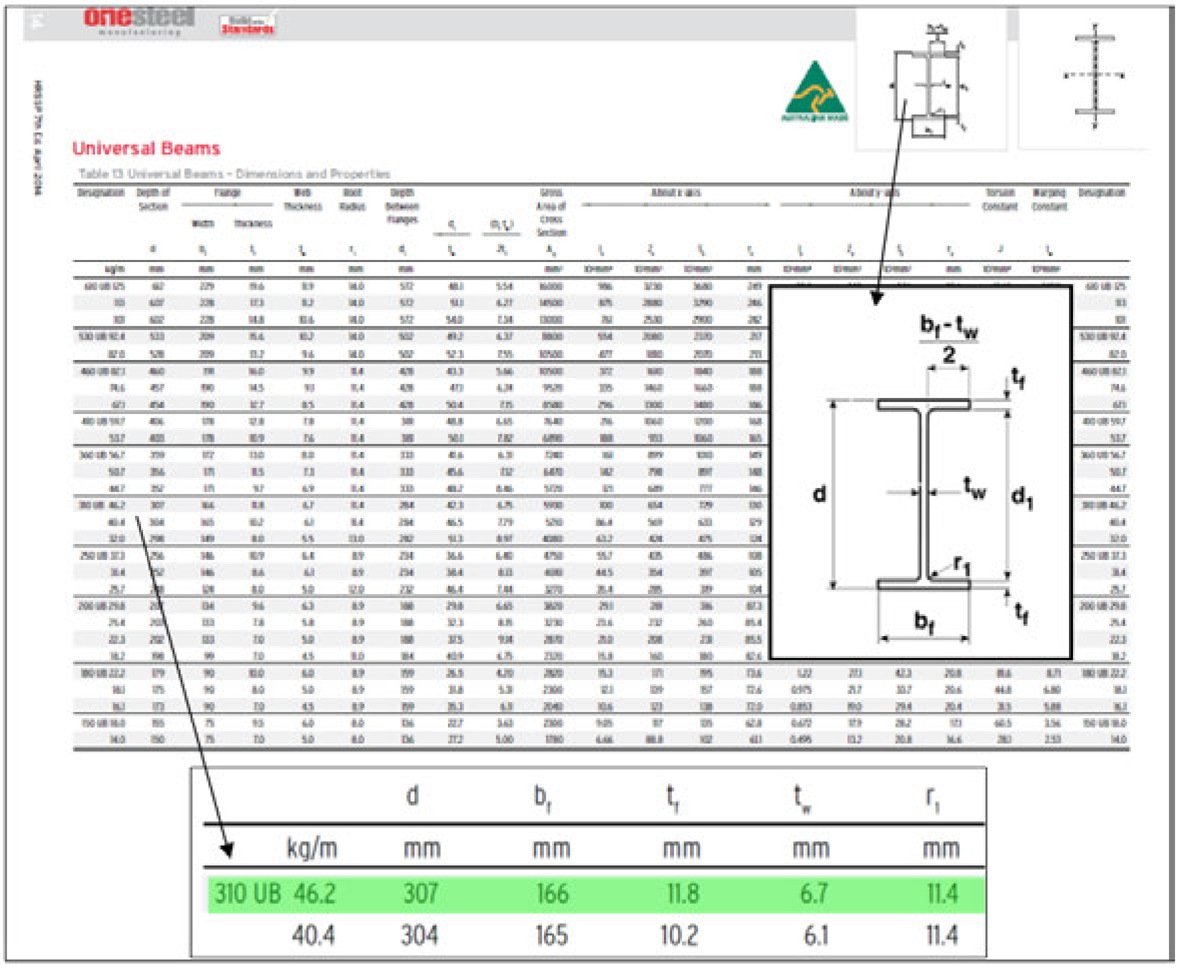


Figure7: Flange and web.

Figure 8 provides an example of a table of dimensions for standard columns as supplied by OneSteel. Let us assume we are observing columns that have the dimensions highlighted above. The flange to flange dimension (D) is 307 mm and the web thickness (t_w) is 6.7 mm.



	d	b_f	t_f	t_w	r_1
kg/m	mm	mm	mm	mm	mm
310 UB 46.2	307	166	11.8	6.7	11.4
40.4	304	165	10.2	6.1	11.4

Figure 8: OneSteel beam/column dimensions (OneSteel, 2014).

6.1.2 Assumption 2: Setup Position

For simplicity, let us assume that it is possible to set up a reflectorless total station such that the web and flange of each column is visible from a single setup position (Figures 9 & 10). It should be noted that this is not always a reality and sometimes more than one setup is required.

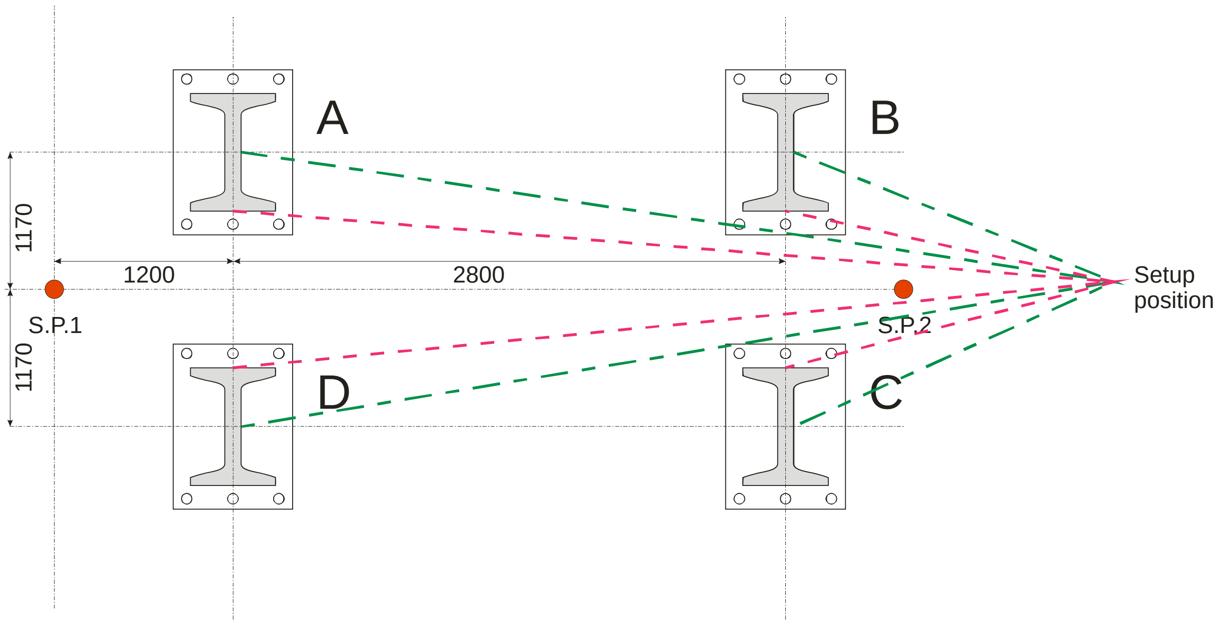


Figure 9: Total station setup.

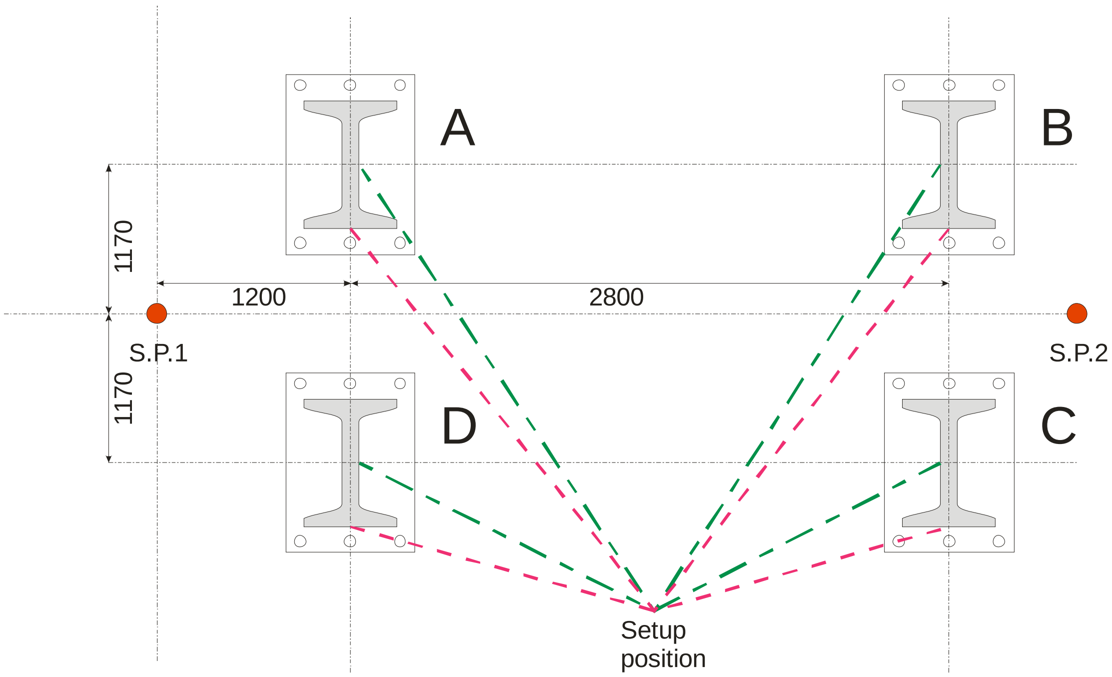


Figure 10: Alternative total station setup.

6.1.3 Assumption 3: Software

The third assumption is that there is access to software capable of calculating chainages and offsets based on a reference line between setout points (S.P.1 and S.P.2 in Figures 9 & 10). This software can be either on board the total station or part of a Computer-Aided Drafting (CAD) package used for post-processing. One example is the ‘Ref Line’ software on board Leica TS15/30 total stations.

6.2 Observation Procedures

The steps followed during observation are straight forward:
1. Set up at a point where you can see the flanges and webs of all columns (or as many as possible because multiple setups may be required).
2. Observe the top of the base plate (or bottom edge of the base plate if practical).
3. Observe the flange of each column as close as possible to the base.
4. Observe the flange of each column as close as possible to the top.
5. Observe the web of each column as close as possible to the base.
6. Observe the web of each column as close as possible to the top.

In this regard, the following should be noted:
- Observations to the web (at the base) are used to calculate the position of the column along the X axis (defined by the line from S.P.1 to S.P.2).
- A comparison of the observations to the web at the base and the web at the top are used to calculate the verticality of the column in the X axis.
- Observations to the flange (at the base) are used to calculate the position of the column along the Y axis (perpendicular to the line from S.P.1 to S.P.2).
- A comparison of the observations to the flange at the base and the flange at the top are used to calculate the verticality of the column in the Y axis.

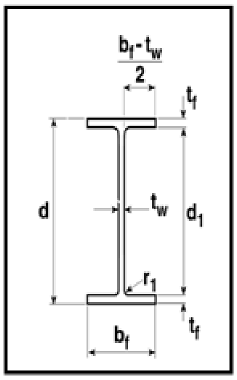
6.3 Calculations

Figure 11 presents an example of results based on a setup similar to Figure 9. Observations to the web and flange are reduced to a horizontal distance (chainage) or horizontal offset from a reference line through the setout points (S.P.1 and S.P.2), then corrected for the observational position relative to the true centre of the column, taking into account which side of the flange and web are observed, the web thickness and the flange width.

The spreadsheet highlights the observations that indicate out of tolerance issues. In this example, the positions of columns B and D exceed tolerances, the verticality of column D exceeds tolerance, and the RL (underside of base) for column C also exceeds tolerance.

Location:	TLO
Date:	12/11/2014
Surveyor:	C. Higgs
Instrument:	TS15 s/n 1234567

		mm
Bf		307
Tw		6.7
Plate		18
RL		10
Vert.	+/-	25
Posit.	+/-	6



Design

	A	B	C	D
Chainage	1200	4000	4000	1200
Offset	-1170	-1170	1170	1170
U/side base (m)	7.450	7.450	7.450	7.450

Colours
Data input
In tolerance
Out of tolerance

NOTE:
+/- Represents observed position relative to centre
t/u Represents top or underside of base plate

Web	A	+/-	B	+/-	C	+/-	D	+/-
Web base	1207	1	4013	1	4002	1	1208	1
Web top	1209	1	4011	1	4001	1	1180	1
X Position	4		10		-1		5	
X Verticality	2		-2		-1		-28	

Flange	A	+/-	B	+/-	C	+/-	D	+/-
Flange base	-1016	1	-1020	1	1011	-1	1008	-1
Flange top	-1017	1	-1018	1	1016	-1	1011	-1
Y Position	1		-4		-6		-9	
Y Verticality	-1		2		5		3	

Plate	A	t/u	B	t/u	C	t/u	D	t/u
RL (m)	7.451	t	7.444	u	7.461	u	7.453	u
RL diff. (mm)	1		-6		11		3	

Figure 11: Extract from spreadsheet calculations.

7 CONCLUDING REMARKS

The remote methodology described in this paper is simple to use and does not require direct access to columns or base plates. It is quick to perform and if onboard software is available the results can readily be determined in the field, a distinct advantage when the construction crew is waiting to grout the base plates or move on to the next phase of construction. This methodology can be easily modified to meet different layout and designs while still being true to the basic assumptions and calculations involved.

ACKNOWLEDGEMENTS

The ideas and methodologies presented in this paper are not my own. They come from shared conversations and shared experiences from a number of surveyors with whom I have worked. Special thanks to Bartek Kocon, a Polish surveyor and friend with whom I worked in Newcastle on the Kooragang Island project, and the team at the Nammuldi Below Water Project in the Pilbara who helped refine the remote methodology presented.

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Re-Markable Roads: When is a Street Fix Fixed?

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ABSTRACT

The original town burghers were keen to formalise the road and street networks of their towns. Hence the introduction of kerb lines via alignment surveys. Alignment gave way to marking street boundaries and placing reference marks: the first being buried concrete blocks, followed 40 years later by pairs of drill hole and wing in concrete kerb and gutter. Subsequently, wholesale loss of these road and street reference marks has led finally to talk of the preservation of those that remained and what to do about the re-marking of existing road boundaries. In a local government area like City of Ryde, where re-development and renewal are rampant, continuing with traditional marking methods may be short-sighted and may have no long-lasting benefits. This paper proposes some policies and examines what City of Ryde is currently doing. The aim of this paper is to garner real support from the surveying industry to assist local government in reclaiming the streets. What happens next is up to you?

KEYWORDS: *Re-marking, roads, corners, surveying*.

1 INTRODUCTION

The first land grants in Ryde, from 1792, all fronted the Parramatta River in a classic rectangular boundary pattern running north-south (Figure 1). River transport was the focus.

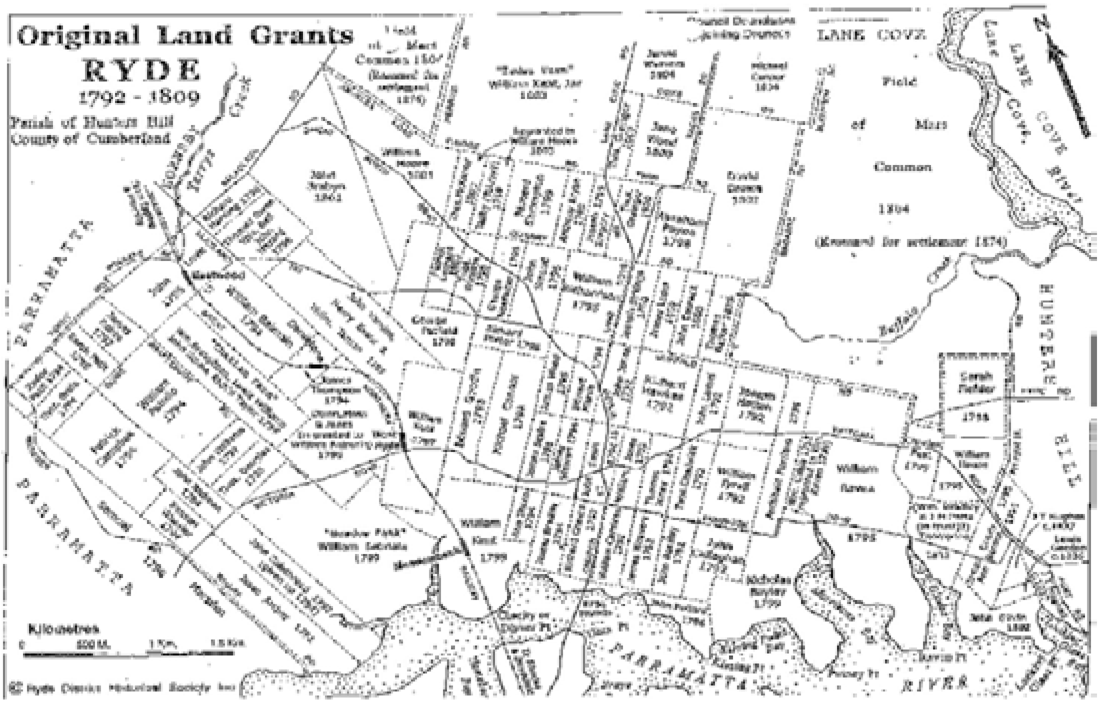

Figure 1: Map of original land grants at Ryde 1792-1809.

During the course of the next 80 years, the main road traversing Ryde ran northwest from the Bedlam ferry wharf at Gladesville to the wharf at Melrose Park. This road did not follow the original road reservations designated in the Crown grants. Many of the colony's early roads became so irregular and poorly defined that the practice of alignment began, i.e. the accurate re-surveying of road boundaries and the leaving of substantial and promisingly long-lasting marks from which to redefine and re-mark road boundaries. Mr Surveyor George Hedgeland's road alignment surveys in Ryde during the 1870s were undertaken to regularise and formalise the boundaries of the existing roads being used. The alignment surveys were actually surveys to re-define streets and roads which were already in use and as such were re-marking the roads. Now, by 2015, the City of Ryde has 845 roads, and except for a handful of RMS-controlled major roads, all are owned and administered by the Council for the City of Ryde (Figure 2).

Figure 2: Map showing the current road layout of Ryde.

2 WHAT DOES A ROAD BOUNDARY MEAN FOR THE SURVEYOR?

Road boundaries have played *the* major role in the integrity of the New South Wales cadastre. Surveyors are taught to determine the road boundary, then to establish the side boundaries of parcels of land and then to redefine the land or to subdivide the land or to create easements etc. This raises the question: If the road boundaries are so important to the cadastre, then how

accurate are they? In answer to that we should know when modern-day survey accuracy was first achieved. The theodolite was the predominant instrument after 1872. Clause 5 of the Regulations for the Employment of Licenced Surveyors issued by the Surveyor General's Department in 1872 reads as follows: "The use of the circumferentor is prohibited…" (Marshall, 2002, p. 30). By 1882, steel ribands were the current instruments in use for linear measurement. It would seem most reasonable to say that surveys were sound as to measurement by 1885, which means that roads created after 1885 should be reliable as to survey. These days what remains after the surveyors depart from the site are boundary marks, reference marks and plans on public record. Note that surveyors are taught to place their cadastral reference marks not on private land, but within the road reservation.

3 HOW AND WHY DOES A ROAD BOUNDARY CHANGE?

Roads are created during the subdivision process, and if surveyors leave sound reference marks there should be no trouble in defining and marking roads in future surveys. So, why do road boundaries shift in definition? By working through the following example we may be closer to an answer. Figure 3 shows the modern day subdivision pattern of six streets in West Ryde. The land was part of a 1799 land grant of 220 acres to William Balmain. The first roads were created in 1881 by DP611, followed in 1883 by DP1095 and DP1257. These three surveys were all carried out by the same surveyor. (It is interesting that these surveys were each completed prior to 1885). 40 years later, in 1921 and 1923, DP611 was totally re-subdivided over three plans by the one surveyor, and three roads were created: Hay, Bennett and Moss Streets. The original road and lot boundaries in DP611 were made at right angles and parallel to Adelaide Street. The re-subdivision of DP611 in the 1920s created three roads parallel to the southern boundary of DP611, which then displayed a swing of 5'45" when compared to Adelaide Street. This southern boundary was defined by the surveyor adopting a line of fencing occupation. The new roads are all in excess of 460 m in length, so minutes count. The side boundaries of the lots created in the re-subdivision were all made parallel to Adelaide Street.

Figure 3: Map showing the road layout of West Ryde in the discussion area.

The three roads south of DP611 (i.e. Darwin, Huxley and Deakin Streets) were created by DP1095 in 1883. Each road was set out parallel to the southern boundary of DP611 and lot boundaries were all created parallel to each other, but at a random angle to the streets.

The first hiccups occurred 40 and 50 years later, with the onset of re-subdivision, when Darwin, Huxley and Deakin Streets were re-fixed parallel to each other but 10 minutes different to the bearing of the southern boundary of DP611. The definition relied on fencing occupation only (Figure 4) and no reference marks were placed until 1939.

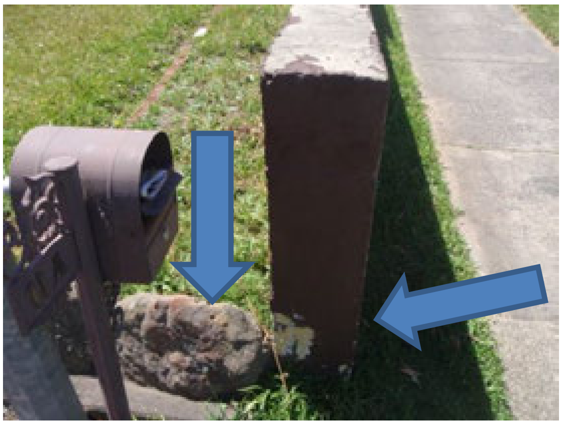

Figure 4: Defining evidence: Corner nail and face of a very old fence.

Since then a constant stream of surveyors has visited this area, undertaken surveys and left their reference marks. Today the road bearings, as currently fixed, are shown in Figure 5. Whatever happened to their parallelism? Darwin Street, now, is even shown with variable width!

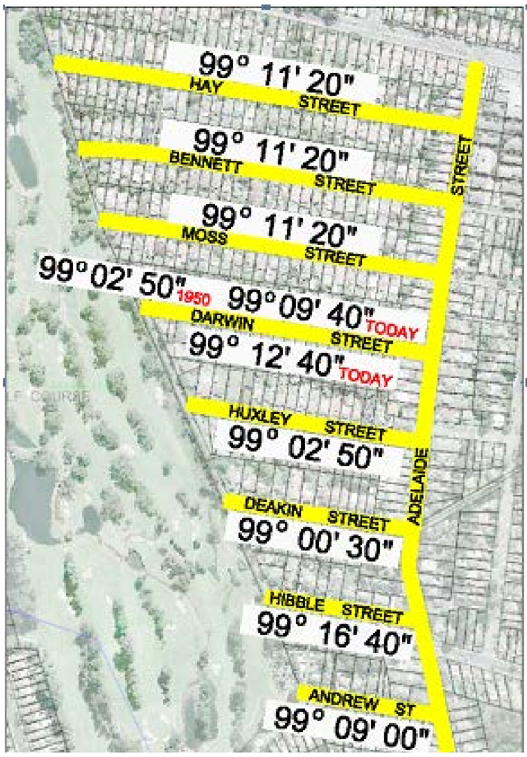

Figure 5: Map showing the current road bearings

Figure 6 illustrates the road layout and subdivision pattern, showing surveying activity. The area shaded yellow comprises the subdivision lots from the original surveys, the area shaded orange comprises plans by compilation on the original lots, and the area shaded pink comprises plans of survey after the original subdivision. Of the original surveys that created these roads and subdivision only one, DP12291 (1923), placed road reference marks: two concrete blocks at the western end of Hay Street. The westernmost concrete block has survived and is still in place today. Since 1924, the northern half, containing Hay, Bennett and Moss Streets, has been subjected to four plans of survey done by three different surveyors! Six reference marks were placed, of which two survive. Since 1924, the southern half, containing Darwin, Huxley and Deakin Streets, has been subjected to 24 plans of survey done by 23 different surveyors. So, is it a case of the more surveyors who work on a road, the more likely it is for the road to change its fix? Land and Property Information (LPI) may find this example re-markable!

Figure 6: Road layout and subdivision pattern showing surveying activity.

4 CAN A STREET FIX BE FIXED AND STAY FIXED?

What can be done differently to circumvent the above results? What can be done in areas away from West Ryde? It is required that suitable survey marks are placed to define road boundaries, i.e. stable marks that will last for a very long time? Would it not be great to have survey marks located, on the boundary, at the end of every road? How easy would boundary surveying be then? How much time would be saved? City of Ryde has been trialling several ideas in an attempt to arrest the widespread loss of marks and monuments which define and fix road boundaries.

4.1 Corner Re-Marks

The idea of placing a galvanised iron (GI) pipe at a street intersection is an old idea (Figure 7) that needs to be re-visited. These roads were created and marked in 1923 and 1926, but it is unknown when the GI pipes were placed.

Figure 7: GI pipes found at two intersections: Beaumont Ave, Denistone and Moira St, Denistone West.

At Ryde, much thought was given to what other type of long-lasting stable mark could be used to indicate a road intersection. Initially, a wooden peg within a metal cover box was tried (Figure 8), but this was considered to perhaps have some ambiguity with a lot corner mark. Consequently, the use of a short steel star picket is now being trialled (Figure 9).

Figure 8: Road intersection at Teemer St, Tennyson Point, where a peg was placed.

Figure 9: Intersection at Parklands Rd, North Ryde, where a star picket was placed.

There are many situations where an intersection point is not suitable for mark placement, e.g. if it falls on a road surface or service installation. In such situations consideration has been given to marking the actual corner at the property boundary. Trials determined that the simplest, cheapest and most readily available mark for the occasion is a standard wooden peg beneath a metal cover box. A peg by itself has little chance of a long life, however combined with a cover box, the whole game changes. In Ryde, if surveyors provide the peg at the road corner, Council will provide the cover box (Figures 10 & 11). In both of these examples, the fencer has arrived long after the corner has been marked and cover-boxed. The new fences have been erected in such a way that the corner marks have been preserved.

At Chatham Road, a new brushwood fence has been placed slightly inside the property boundary, which allows clear and easy access from the road to the mark (Figure 12). It should be noted that the fencer has kindly checked out a section of the bottom rail to enable lid opening.

Many residents are excited by the idea of having their lot corner preserved. Then there is the question of what happens if the boundary needs to be fenced? Substantial structures, such as a brick fence, block fence or dressed stone fence are declared in a Development Application (DA) and given a DA condition, which specifies that the face of the fence be erected on the boundary line, the position being verified by the site surveyor after construction. Two occasions have occurred (Figure 13) when City of Ryde provided the boundary information and the new rendered fences were erected on the road boundary lines. These new occupations are likely to be in existence for decades, and will be especially useful in the future, knowing the surveyors' penchant for adopting occupations at the ends of roads when reference marks have disappeared.

Figure 10: Road widening at Balaclava Rd, Eastwood, where a peg was found.

Figure 11: Corners at intersection of Chatham Rd and Dickson Ave, West Ryde.

Figure 12: New brushwood fence is erected inside the property line and allowance made for lid opening.

Figure 13: Samuel St, Ryde and Ford St, North Ryde.

4.2 Reference Re-Marks

Since the demise of alignment marks, old stone kerbs and concrete blocks which were placed to indicate the road boundaries (Hallman, 1973; Marshall, 2006), great reliance has been placed on drill hole and wing reference marks placed in concrete: concrete kerbs, concrete gutters, concrete footpaths, concrete driveways and concrete drainage structures. Such concrete provides a great medium for traverse stations and base marks but is totally unsuitable for ensuring a long-lasting survey reference mark. Figure 14 shows the stockpile of concrete removed from the roads in the City of Ryde in just one year. This pile of concrete from 2014, comprising 6,400 m^3, has now been crushed and recycled as roadbase. The beginnings of the 2015 stockpile are shown in Figure 15, and already it consists predominantly of kerbs.

Use of buildings for reference marking is, once again, not a new idea but one well worth re-inforcing within the surveying industry. Ever since the original alignment surveys, the old Water Board detail surveys and the City of Sydney detail surveys (Cadogan, 1996), buildings have been used as monuments to aid in the definition of road boundaries. In DP395460 (1955) the surveyor placed an “RM Nail in Bk Wall” (Figure 16) to reference a bend in Marlow Avenue, Denistone.

Figure 14: Stockpile of concrete removed from the roads of one council in one year (2014).

Figure 15: Stockpile of concrete removed from the roads of the same council in one month.

Figure 16: Reference mark using GI nail in brick wall.

This mark has subsequently been used by surveyors in subdivision and redefinition surveys in 1975, 2004, 2007 and 2014 to re-establish the road boundary. That is a life of 60 years so far. What are its chances of survival for another 60 years? Unfortunately, slim! This residential

area is currently undergoing a vibrant renewal, with new dwellings and duplexes rapidly replacing the first homes. Which buildings, today, are likely to be suitable as the next repository of a survey reference mark? Suggestions include new dwelling houses and new appartments (each of which would expect a lifetime of many decades), government buildings and heritage listed buildings (Figure 17). Obviously, free and unimpeded access to the mark must be a consideration.

Figure 17: Very old Survey Bench Mark in face of a heritage building in Parramatta.

During a recent conversation had with a boundary surveyor, who was preparing a site in Ryde for a major redevelopment, it was impressed upon him that the placing of survey reference marks in concrete kerbing was perhaps not the best option. “So, what am I supposed to do, place GI pipes?” Actually, this is a very viable option. At this site the ageing kerbing and footpath will probably be totally replaced during the redevelopment. Obviously, any reference marks placed in concrete during the boundary definition phase would probably not last two years! A GI pipe (with cover box placed by Council) on the other hand, positioned above existing telecommunication (telco) and electricity ducts has a great chance of surviving for many years. Service trenches are unlikely to be re-dug as surplus ducts are placed during the initial trenching. In fact, it is quite a nice twist to think that GI pipe reference marks could go back into the space where telco was so destructive of reference marks prior to 1960.

4.3 SCRIVs

For decades, a handful of surveyors have been taking advantage of large telco pits and large electricity pits as suitable sites in which to place survey marks (Figure 18).

Figure 18: State Survey Mark in telco pit, reference mark in pit, and Bench Mark in pit.

The huge advantage of the large telco pit, apart from its size and bulk (there is more concrete comprised in such a pit than in a SCIMS Permanent Mark), is that the pit is virtually untouchable. It is extremely rare that any civil works even consider moving or adjusting a telco pit. The telecommunication companies themselves are loathe to adjust an existing pit, preferring instead to construct a totally new pit adjacent and on line. In fact, the telco companies have no maintenance program in place for their older network pits. Many of these pits have already been around for 60 years and there is every reason to think that they will continue on for many more decades. However, there is one major hazard when utilising the concrete on the top of a large telco pit: the possibility that any footpath upgrade will tamper with or remove the top edge of the pit. Paving work has been interfering with the tops of large telco pits for years (Figure 19).

Figure 19: Old concrete path laid up to very edge of the metal lid frame.

Other examples (Figures 20 & 21) also show why the concrete edge of a telco pit may be unsuitable. It is becoming common practice to remove the concrete edging and pave right up to the pit lids. These town-centre paving works were all done in 2014 and are clearly a sign of the future.

What remains visible and viable after the paving work is finished is the metal frame which holds the pit lids in place. City of Ryde has been trialling the placing of a quick, cheap and obvious mark into one corner of the metal frame of large telco pits. This mark is a 3 mm diameter drilled hole, into which is hammered a humble clout (Figure 22).

Figure 20: Preparation of a telco pit during a footpath upgrade and the finished result.

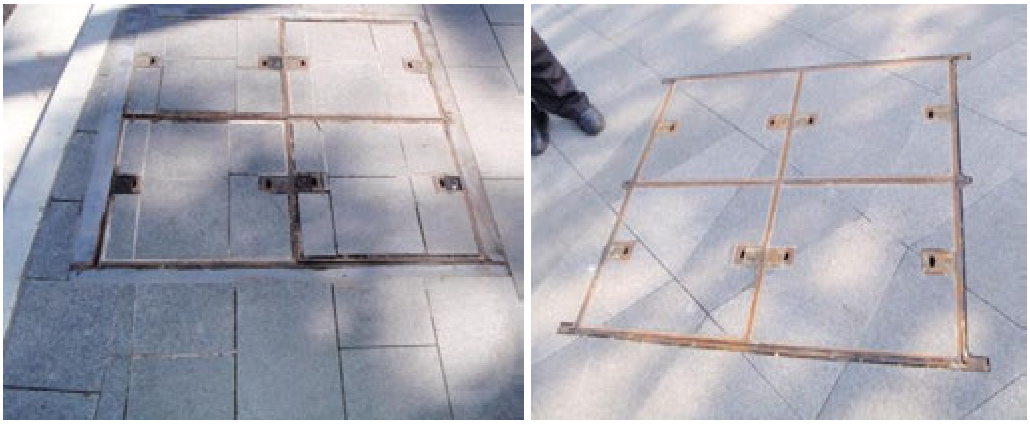

Figure 21: Waterloo Rd, Macquarie Park and Parramatta's Centenary Square.

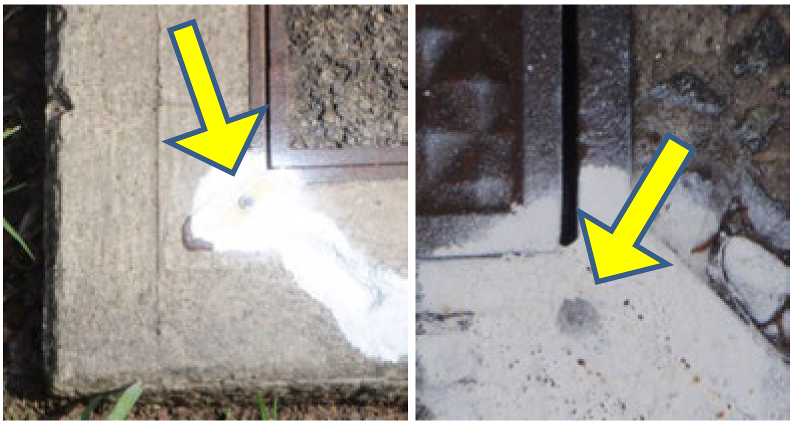

Figure 22: Two examples of SCRIVs placed in lid frames.

City of Ryde believes that Survey Control and Referencing Is Vital (SCRIV), and what we are trying to achieve is a system of marking that has a great chance of lasting more than 60-70 years, or at least three generations of surveyors. It is only fitting that our special mark is called a SCRIV, named after Mr Surveyor Charles Robert Scrivener, who was so keen to leave long-lasting marks. He gave us such impressive monuments as big stone alignment posts, Burrenjuck Dam, the Murrumbidgee Irrigation Area and Canberra. We believe that a SCRIV has a much better chance of long-term survival than marks placed in concrete kerbs or concrete paths (cf. Figures 14 & 15). Unfortunately, not every road has a usable large telco pit.

Surveying students at the University of New South Wales (UNSW) have had an inkling of the use of SCRIVs for years now as evidenced by a drilled hole found in a large telco pit on campus near the Civil Engineering Building (Figure 23). For the naysayers, the author has managed to find one example in Ryde where a SCRIV would have been a failure, noting that the entire metal frame has been broken away (Figure 24).

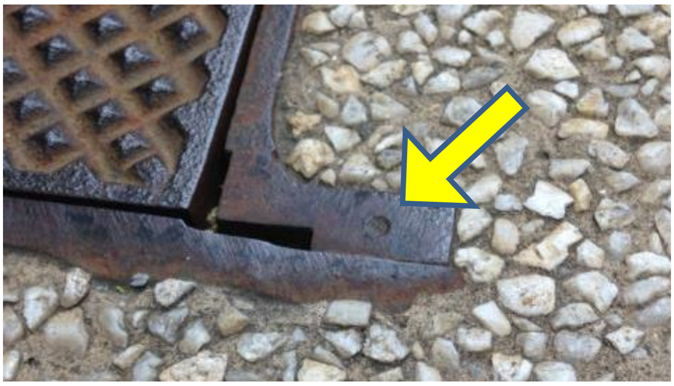

Figure 23: Drill hole found placed in lid frame at UNSW.

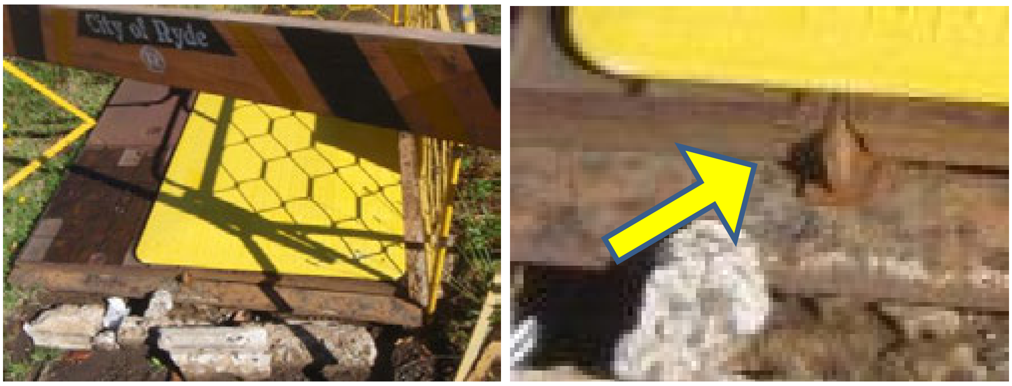

Figure 24: Telco pit with major damage, including missing concrete and metal frame.

4.4 Control Re-Marks

A recent audit of SCIMS control marks throughout Ryde indicated that 149 marks have been destroyed (out of a total of 1,202 marks placed). An investigation of these lost marks was carried out to determine the age or lifespan of each mark (i.e. when was the SCIMS mark placed). Deciding to adopt 20 years as a starting point, a tally was made from the lost marks. 101 marks (i.e. two thirds) were younger than 20 years. How many of these marks were PMs? How many of these marks were brass SSMs? The staggering result was that one lost mark was a PM, and 100 lost marks were SSMs! Obviously the reliance and practice of placing SSMs in concrete kerb and gutter and concrete footpaths is rapidly failing in an area such as Ryde (again, see the stockpiles of concrete in Figures 14 & 15). Enquiries at LPI uncovered another important statistic. SSMs are sold to surveyors in urban areas at a rate of 10-to-1 compared to PMs. This suggests to me, that with SSMs having such a very short life, that up to 90% of LPI resources, when it comes to SCIMS marks, is being directed towards coordinating and levelling SSMs when a much better outcome would be to spend 90% of the LPI effort on PMs! This leads to another observation that was made in the City of Ryde: All of the 149 destroyed SCIMS control marks had been lost from sites within the road reserve. This leads to the next suggestion: Only PMs should be placed in the road reservation, while SSMs should be placed in parks.

4.5 Marks in Parks

In parks there is more likelihood of having a suitably clear site for Global Navigation Satellite System (GNSS) observations (Figure 25). In a heavily treed area, like Ryde, most streetscapes do not provide ideal locations for precise GNSS positioning. LPI is moving in this direction

with its placement of marks for major control networks. Losses of SCIMS marks in road reservations have become so frequent that the control network being established in Carlingford is utilising this notion of marks in parks.

Figure 25: Ryde Park showing easy access and clear skies.

The City of Ryde survey team is likewise using parks as sites for control marks (Figure 26). Galvanised star pickets at Granny Smith Memorial Park were placed in conjunction with the building of and upgrading of a new children's playground.

Figure 26: SS79424 placed 20 years ago at Portius Park, East Ryde, and galvanised star picket placed in 2014 at Granny Smith Memorial Park.

5 CONCLUDING RE-MARKS

Fixing the boundaries of roads in such a way that the definition lasts a very long time necessitates a wholesale change of thinking as to the methods surveyors now use to mark the road boundaries. Which way should the surveying industry head when it comes to corner marking, reference marking and control marking? This paper proposes that a simple wooden peg with metal cover box can have a long life expectancy, that marking of road intersections is worthwhile and should be encouraged to continue, that placing survey reference marks in concrete kerb and gutter, concrete footpaths, concrete driveways, concrete drainage structures or concrete edging is a short-sighted practice and should be actively discouraged, that State Survey Marks should leave the road reservation and be placed on community land in parks, and that a clout or pin placed in the metal frame of the lid support on a large telco pit may be

a suitable alternative as a reference mark when it comes to fixing a street. What happens next is up to you!

ACKNOWLEDGEMENTS

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Cadastral Integrity in NSW: Avoiding Slippery Situations through Examination, Investigation and Supervision

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ABSTRACT

In New South Wales, land and mining surveyors are registered under the provisions of the Surveying and Spatial Information Act 2002 ("the Act"). The Act is administered by the Board of Surveying & Spatial Information (BOSSI) of NSW, a statutory authority created under the Act. Pursuant to Section 2A of the Act, one of BOSSI's objects is "to provide for the maintenance of a State cadastre and ensure its integrity". Three ways to achieve this object are the examination of candidate surveyors, the investigation of complaints against registered surveyors and ensuring that the supervision of unregistered persons by registered surveyors is robust and accountable. This presentation includes case studies of investigations, errors made by candidate surveyors and the expectations of the Board in regard to supervision.

KEYWORDS: *Cadastral surveying, BOSSI, investigation, examination, supervision.*