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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.

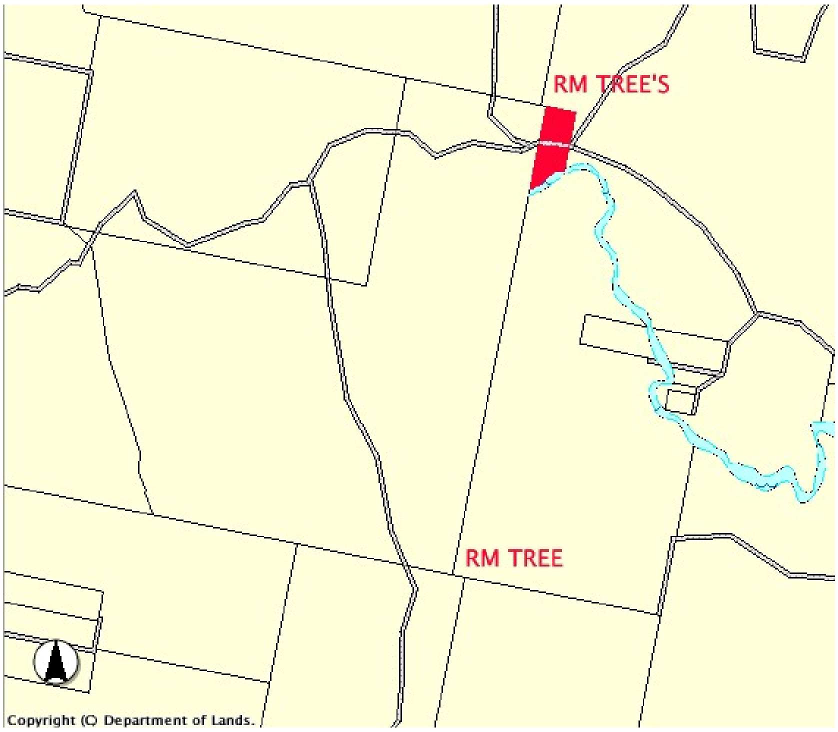

Figure 1: Woods Reef.

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.

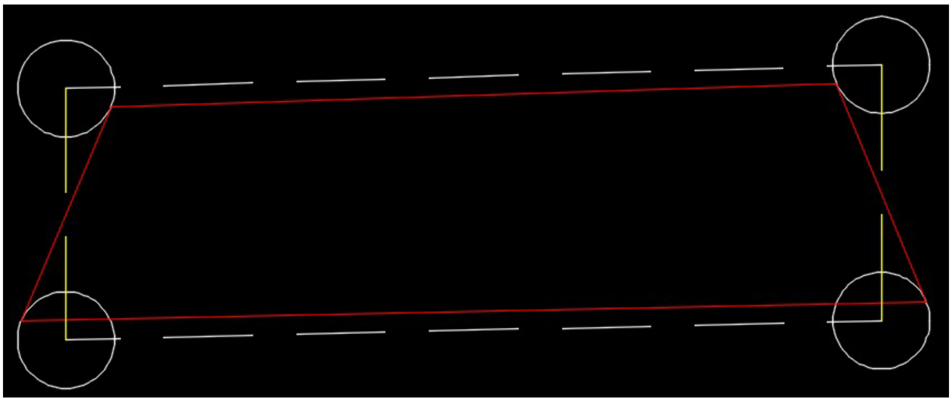

Figure 3: Exaggerated example of azimuth swing concept.

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.

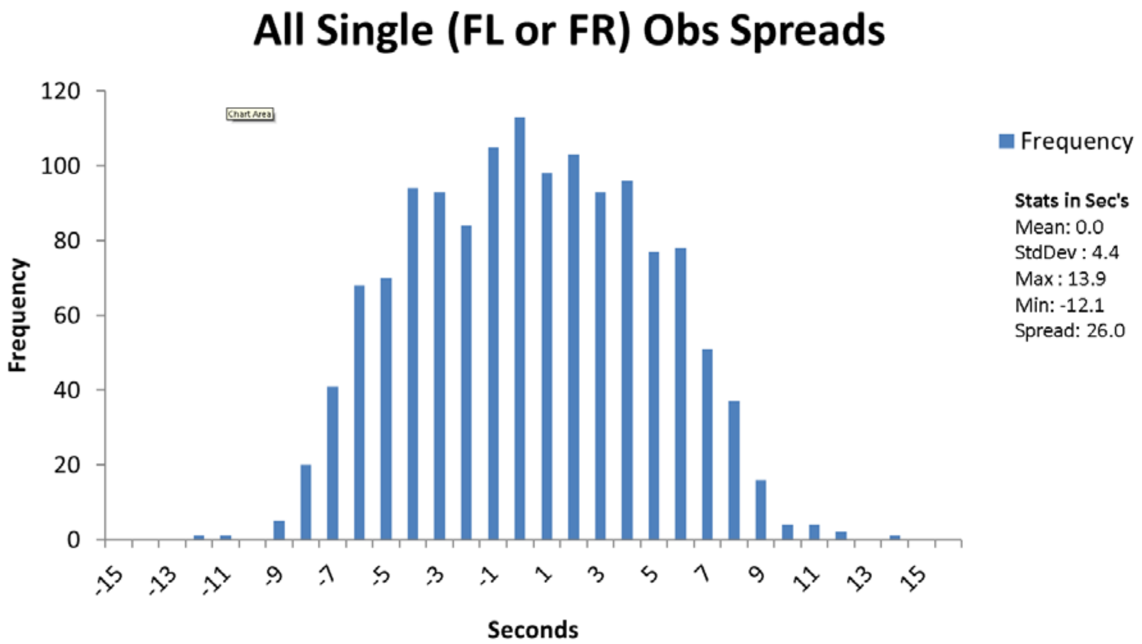

Figure 4: Spread of single observation.

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?

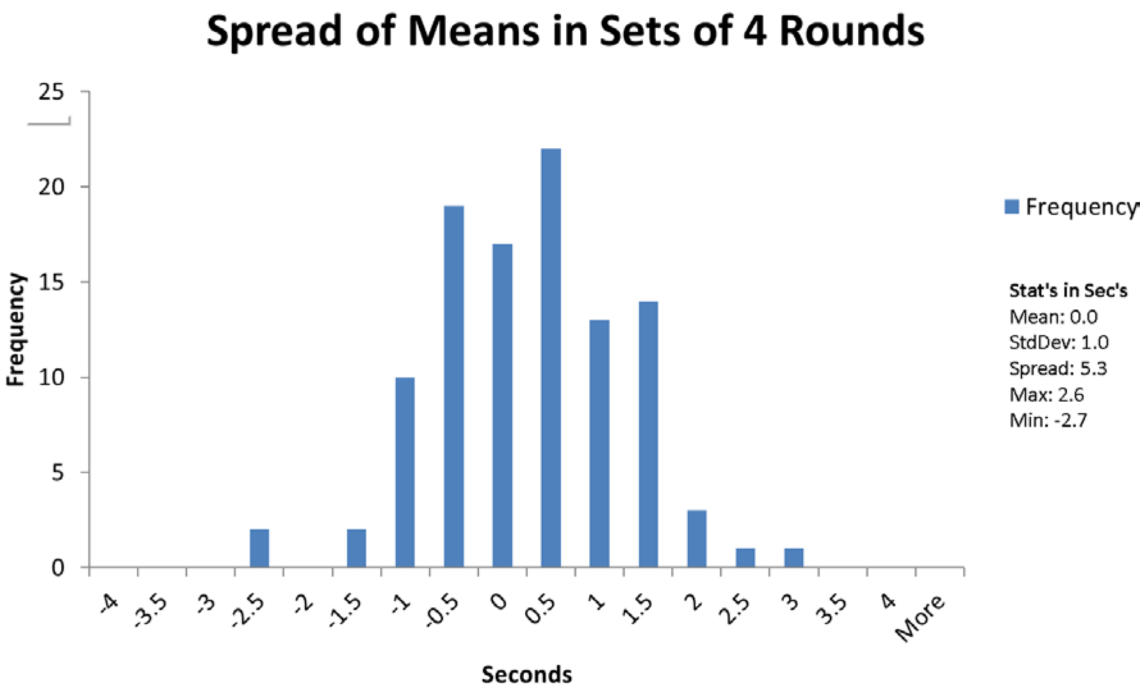

Figure 5: Spread of means in set of 4.

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.

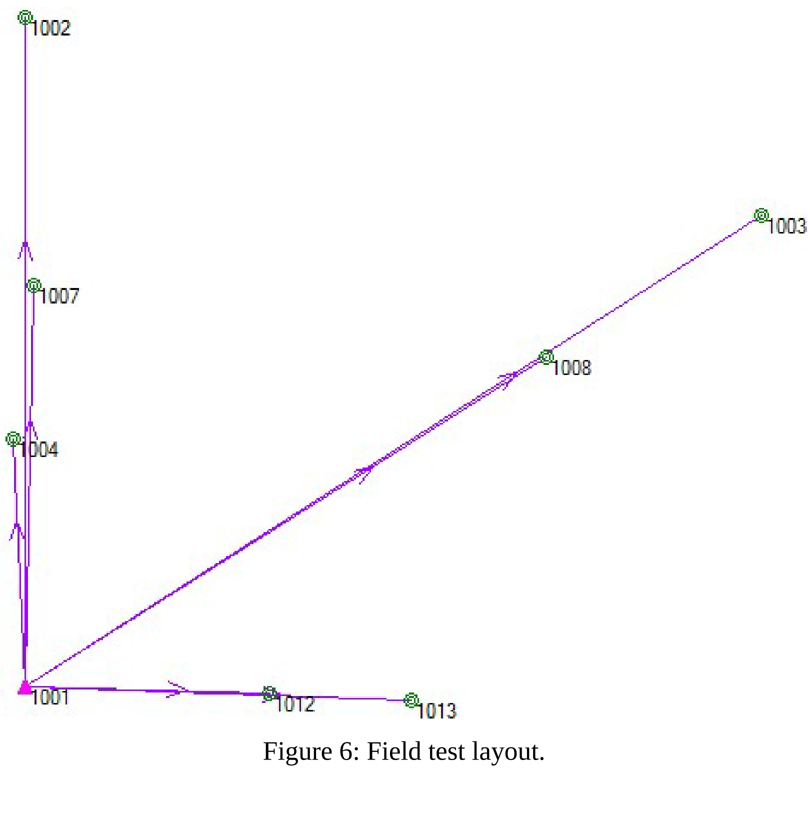

Figure 6: Field test layout.

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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