A New Redefinition of Geodetic and Plane Coordinates on UTM Geodetic Markers

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ABSTRACT

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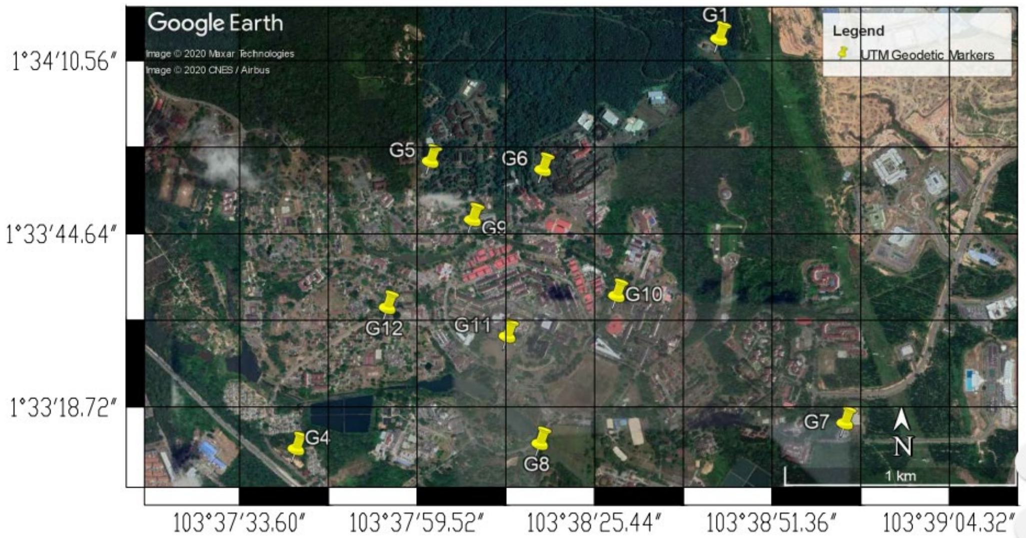
The entire Peninsular Malaysia is situated on Sunda Tectonic Plate, which is subjected to motion at a prominent rate. All the geodetic infrastructures, including geodetic markers, benchmarks, Continuous Operating Reference Stations and Cadastral Reference Marks have moved away from their original position, and their existing coordinates are no longer reliable and secure to use. There are 10 geodetic markers around UTM that are subjected to the issue above. This study aims to redefine new geodetic and plane coordinates on UTM geodetic markers. Two units of Trimble NetR9 Geodetic Type Receiver are used to execute 3D GNSS Control Network on geodetic markers as well as Standard Benchmarks. Receiver Independent Exchange data of Continuous Operating Reference Stations and gravimetric geoid of MyGeoid model are retrieved from Department of Survey and Mapping Malaysia. Trimble Business Center, Golden Surfer 8 and StarNet are used as post-processing, geoid interpolation and one-dimensional network adjustment software, respectively. New sets of geodetic and plane coordinates along with orthometric heights are produced for these 10 geodetic markers. Eventually, UTM geodetic markers are tied to Geocentric Datum Malaysia 2000 (2016) and National Geodetic Vertical Datum, providing reliable horizontal and vertical reference for land surveying work to fulfil both industrial and educational purposes.

Keywords: *Coordinate redefinition, geodetic coordinate, plane coordinate, geodetic marker*

INTRODUCTION

Geodetic infrastructures are a set of physical monuments that realise geodetic datum. In national scale, two of the most important geodetic infrastructures are Malaysia Real Time Kinematic Network (MyRTKNet) and Levelling Benchmarks, which constitute Geocentric Datum of Malaysia 2000 (GDM2000) based on the ITRF2000 at epoch 2000.0 and National Geodetic Vertical Datum (NGVD), respectively. Universiti Teknologi Malaysia (UTM) also owned a set of geodetic infrastructures known as geodetic markers. There are 10 geodetic markers around UTM as shown in Figure 1.

Practitioners did not use these geodetic markers due to its outdated datum, which is Peninsular Malaysia Geodetic Scientific Network 1994 (PMGSN94) that connected to the first version of World Geodetic System 1984 (WGS84) (DSMM, 2009a; DSMM, 2009b). However, current practice is GDM2000 (2016) based on the ITRF2000 at epoch 2000.0 (Muhammad Afiq, 2019; Che Amat, 2020). In relation to that, the plane coordinates are in the old version, which is Malaysia Rectified Skew Orthomorphic (MRSO). The main reason that geodetic markers are not being used by practitioners is because there is no reliable plane coordinate to fulfil the mapping purpose. Moreover, there is no accurate height information regarding these geodetic markers but only their ellipsoidal heights and orthometric heights derived from Earth Gravitational Model 1996 (EGM96). Figure 2 shows an example of a geodetic marker monument for G11 which is located at Helipad, UTM.



Source: (Modified from Google Earth, 2020)

Figure 1. Distribution of 10 geodetic markers around UTM

Figure 2. Marker G11 located at Helipad, UTM

Due to the dynamics of the earth, geodetic infrastructures will gradually displace. When the location of reference stations is shifted, geodetic datum must be revised to compensate the dislocation (Gill *et al.*, 2016). As a country that is located on Sunda Tectonic Plate, Malaysia experienced such a situation, with well-known examples the earthquakes that occurred in 2004, 2005, 2007 and 2012. Consequently, GDM2000 is not geocentric anymore (Zulkifli, Din and Omar, 2019). All geodetic infrastructures owned by the Department of Survey and Mapping Malaysia (DSMM) have shifted from their original position and their reliability has thus been degraded. Nevertheless, DSMM has counteracted by carrying out static GNSS campaign from 2006 to 2009 to revise the coordinates of all MyRTKNet CORS and eventually produced a list of CORS coordinates, known collectively as GDM2000 (2009) (DSMM, 2009a; DSMM, 2009b). Recently in 2016, DSMM has made a new revision for geodetic datum in Malaysia, namely the GDM2000 (2016). This is to ensure that Malaysian geodetic datum is compatible to the current development of GNSS infrastructure and also to maintain the geocentric datum (Muhammad Afiq, 2019; Che Amat, 2020). As mentioned by Abidin *et al.* (2015), Bawa *et al.* (2019) and Rabah *et al.* (2019), many countries worldwide had subsequently maintained, revised and updated their national geodetic datum to assure the reliability of land survey works that are being carried out within their nations.

Unlike DSMM, UTM never revise the coordinates of geodetic markers. The coordinates published by Laboratory of Geodesy and Astronomy (MGA), UTM are still tied to old geodetic datum, which is PMGSY 94. The orthometric heights are not consistent with NGVD, given that the geoid model used in deriving orthometric heights is EGM96 instead of MyGeoid, which is the geoid model required for GNSS heighting in Malaysia (DSMM, 2005) to determine orthometric heights with accuracy of 5cm (Jamil, 2011). The coordinates of geodetic markers published by MGA are summarised in Table 1.

Table 1. Existing coordinates in the geodetic datum of PMGSN94 published by MGA

Station	Latitude	Longitude	Ellipsoidal height (m)
G1	1°34’10.878367”	103°38’42.137728”	142.2738
G4	1°33’13.790142”	103°37’44.143816”	47.4657
G5	1°33’54.450184”	103°38’02.745990”	47.1259
G6	1°33’53.243354”	103°38’18.309356”	52.9572
G7	1°33’17.226695”	103°39’00.425929”	22.3003
G8	1°33’14.305735”	103°38’17.839988”	21.9004
G9	1°33’46.180907”	103°38’08.513461”	29.7453
G10	1°33’35.372172”	103°38’28.566394”	24.7870
G11	1°33’29.644851”	103°38’13.343269”	41.4441
G12	1°33’33.691507’	103°37’56.666119’	32.6383

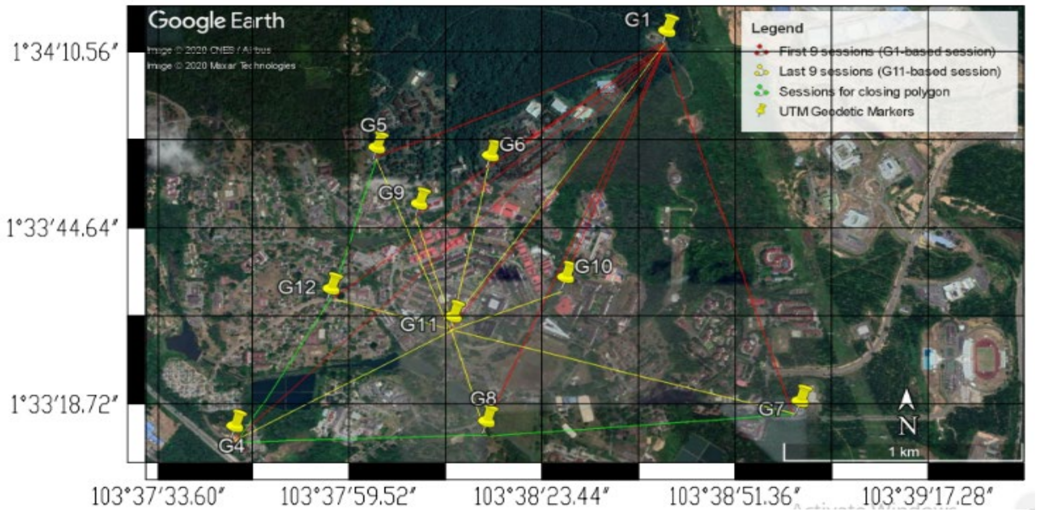
Another issue of geodetic markers is that the existing published coordinates by MGA do not have new map projection systems, which are RSO Geocentric and Cassini-Soldner Geocentric. These two map projection systems are very important in terms of surveying application, which are for mapping and cadastral purposes, respectively. To be qualified as good geodetic infrastructures as acknowledged by practitioners, UTM geodetic markers are required to have these two map projection systems.

Therefore, the purpose of this study is to redefine the geodetic and plane coordinates on UTM geodetic markers by carrying out static GNSS observation using two units of Trimble NetR9 Geodetic Type Receiver. Trimble Business Center (TBC) is used as post-processing software. All geodetic markers will have new geodetic coordinates in GDM2000 (2016) geodetic datum and new plane coordinates in both RSO Geocentric and Cassini-Soldner Geocentric. New orthometric heights by connecting with National Geodetic Vertical Datum (NGVD) for all geodetic markers are published along with the coordinates. The outcomes of this study will benefit students as they have more geodetic markers available to carry out their research and practical works. Practitioners from industry are also welcome to utilize these geodetic markers because the updated coordinates are tied to the latest geodetic datum of GDM2000 (2016) and reliability is guaranteed.

DATA AND METHODS

3D GNSS Control Network Design

Since only two units of Trimble NetR9 Geodetic Type Receiver are available, the network design is based on a single-baseline per session approach as shown in Figure 3. Each session lasted for two hours and the data was recorded at 10 second interval.



Source: (Modified by Google Earth, 2020)

Figure 3. The map showing network design that is executed in this study

As can be seen from Figure 3, there are three types of single baseline: G1-based, G11-based and closing polygon baseline. G1 and G11 are chosen as base markers because of their good surrounding environment and open sky condition, which is satisfying long-occupying criteria, given that one of the two units of Trimble NetR9 receiver is occupied for a longer duration during each day. Detailed discussion is included in subsection 2.3 Static GNSS Observation.

Note that MyRTKNet CORS are not yet included in network design. The inclusion of CORS RINEX data is discussed in subsection 2.6, where the post-processing approach is briefly explained and discussed.

Field Equipment Setup

Two units of Trimble Net R9 Geodetic Type Receivers are used to carry out static GNSS Observation based on network design stated in subsection 2.1. Since NetR9 is a geodetic type receiver, the receiver unit and antenna unit are separated. As for the part of the antenna, Trimble Zephyr 3 ground plane antenna is used. This combination of field equipment (Trimble NetR9 + Trimble Zephyr Geo 3) is the best GNSS equipment owned by MGA, despite the busier setup during the field observation preparation. Figure 4 shows the field setup of this study.

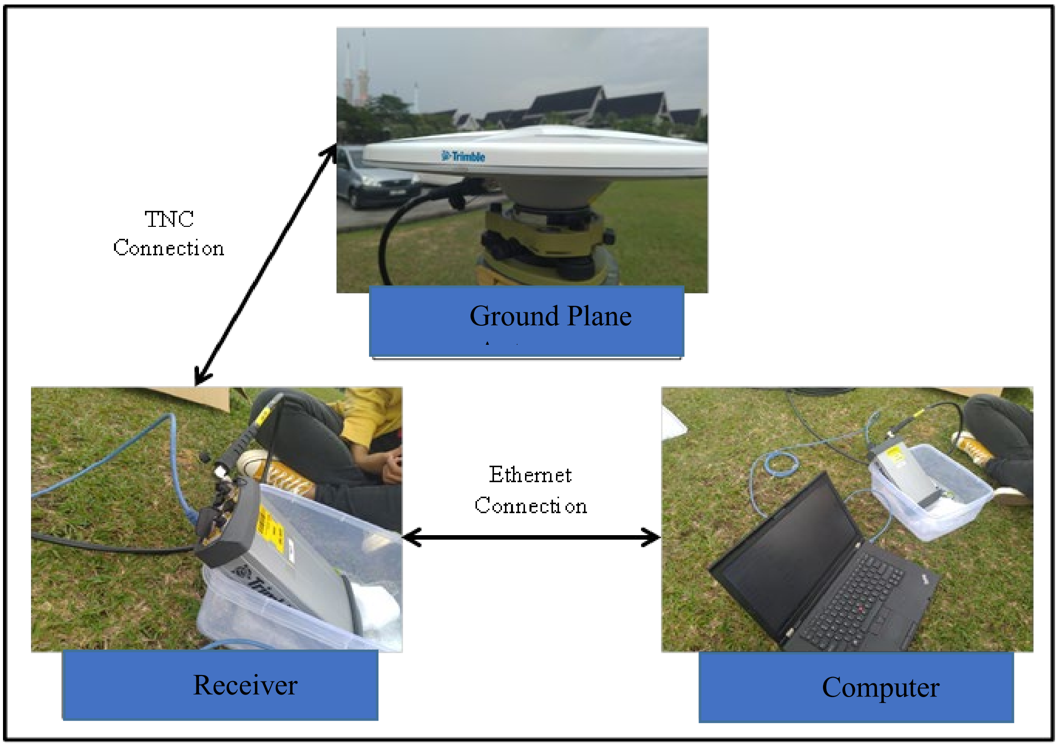


Figure 4. Field setup of this study. Connection to the computer is for downloading RINEX files

Static GNSS Observation

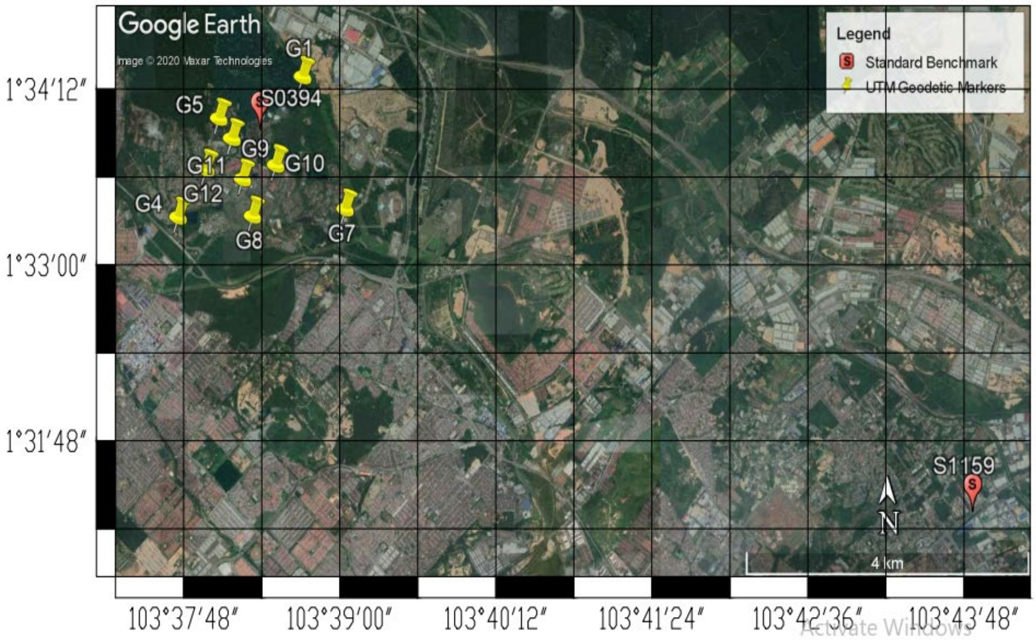
The 3D GNSS control network designed in subsection 2.1 is executed by carrying out static GNSS observation from 13 January 2020 to 19 January 2020 as shown in Table 2.

Table 2. Session execution date, time and instrument height information

Date	Session	Time	Base Instrument height (m)	Rover instrument height (m)
13/1/2020 (Monday)	G1-G5	0836 to 1036	1.115	1.084
	G1-G9	1051 to 1251	1.115	1.151
	G1-G6	1306 to 1506	1.115	1.030
	G1-G10	1523 to 1723	1.115	1.218
14/1/2020 (Tuesday)	G1-G7	0848 to 1048	1.206	0.965
15/1/2020 (Wednesday)	G1-G4	0829 to 1029	1.103	1.229
16/1/2020 (Thursday)	G11-G1	0833 to 1033	1.125	1.126
	G11-G5	1055 to 1255	1.125	1.132
	G11-G9	1312 to 1512	1.125	1.214
	G11-G6	1535 to 1735	1.125	1.205
17/1/2020 (Friday)	G11-G1	0830 to 1030	1.045	1.107
	G11-G10	1052 to 1252	1.045	1.295
	G11-G7	1313 to 1513	1.045	1.138
	G11-G4	1533 to 1733	1.045	1.168
18/1/2020 (Saturday)	G12-G1	0830 to 1030	1.079	1.130
	G12-G5	1052 to 1252	1.079	1.155
	G12-G11	1321 to 1521	1.079	1.116
	G12-G4	1542 to 1742	1/079	1.120
19/1/2020 (Sunday)	G8-G1	0846 to 1046	1.240	1.044
	G8-G7	1109 to 1309	1.240	1.184
	G8-G11	1323 to 1523	1.240	1.150
	G8-G4	1540 to 1740	1.240	1.190

Fieldwork for Relative GNSS Heighting

This study utilized relative GNSS heighting instead of absolute GNSS heighting because the relative heighting method is a more reliable method as emphasized in many previous studies, most notably by Talib *et al.* (2014) and Zakaria *et al.* (2019). Two Standard Benchmarks (SBM) owned by DSMM, S0394 and S1159 are selected for the purpose of relative GNSS heighting. Figure 5 shows the location of S0394 and S1159, with S0394 located inside UTM campus, while S1159 is located 11 kilometers away from campus area.



Source: (Modified from Google Earth, 2020)
Figure 5. The map showing location of S0394 and S1159

SBM are used instead of typical Benchmarks (BM) because the monuments of SBM are designed to provide extra stability and reliability as compared to typical BM. SBM are usually located inside public infrastructure areas such as schools or police stations. Typical Benchmarks are located near the main road hence they are relatively less stable and may shift from time to time as compared to SBM which are less disturbed.

Table 3 shows the description of both SBM as taken from DSMM website while Table 4 summarizes the fieldwork detail of relative GNSS heighting.

Table 3. Description of both SBM selected in this study

SBM	Latitude	Longitude	Orthometric height (m)
S0394	1°33'56"	103°38'30"	62.215
S1159	1°31'19"	103°43'50"	34.870

Table 4. Relative GNSS heighting date, time and instrument height information

SBM	Date and Time	Instrument height (m)
S0394	0846 to 1146, 20 January 2020 (Monday)	1.246
S1159	0936 to 1336, 1 Mac 2020 (Sunday)	1.432

Figure 6 shows the surrounding environment of S0394 and S1159 during observation. The observation at S0394 was initially set for 3 hours (1 hour longer than previous session) because it is located behind of a building and subjected to severe multipath, which will affect the positioning accuracy (Han, Tang and Deng, 2019; Braasch, 2017). After it was discovered that 3 hours data all are very bad, the observation at S1159 is prolonged to 4 hours. The post-processing approach for relative GNSS heighting is discussed in subsections 2.6 and 2.7.

Figure 6. Observation at S0394 (left) and S1159 (right)

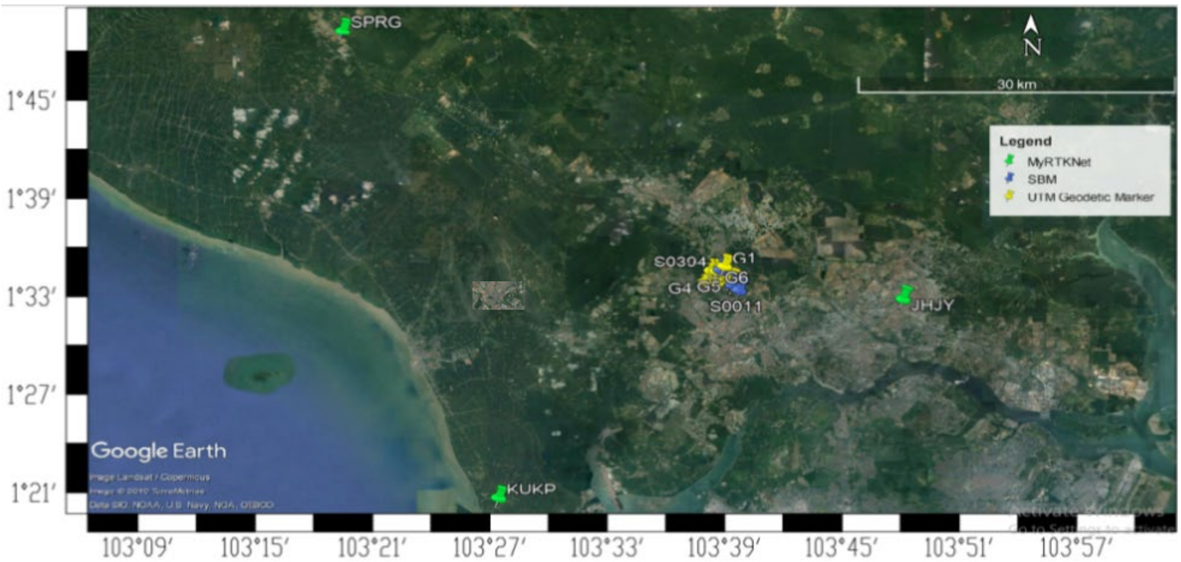
Data Acquisition from DSMM

RINEX files of CORS

Figure 7 shows the location of 3 CORS chosen in this study, which are JHJY, KUKP and SPGR. These 3 CORS are the closest to the study area among all the CORS and hence they are chosen to become constraining stations in this study.

Gravimetric Geoid

The gravimetric geoid data obtained from DSMM is shown in Table 5 and visualised in Figure 8. It consisted of gravimetric geoid values at twelve equally spaced points that covered the entire UTM campus area. The exact values of gravimetric geoid at 10 geodetic markers are obtained by interpolation using Golden Surfer Software after TBC produced their latitude and longitude.



Source: (Modified from Google Earth, 2020)

Figure 7. Three MyRTKNet CORS chosen in this study: JHJY, KUKP and SPGR

Table 5. Gravimetric geoid data acquired from DSMM

Longitude	Latitude	Gravimetric Geoid Value (m)
103.6100000	1.5800000	6.378
103.6100000	1.5633333	6.380
103.6100000	1.5466666	6.384
103.6266667	1.5800000	6.448
103.6266667	1.5633333	6.452
103.6266667	1.5466666	6.457
103.6433334	1.5800000	6.519
103.6433334	1.5633333	6.527
103.6433334	1.5466666	6.533
103.6600001	1.5800000	6.590
103.6600001	1.5633333	6.600
103.6600001	1.5466666	6.607

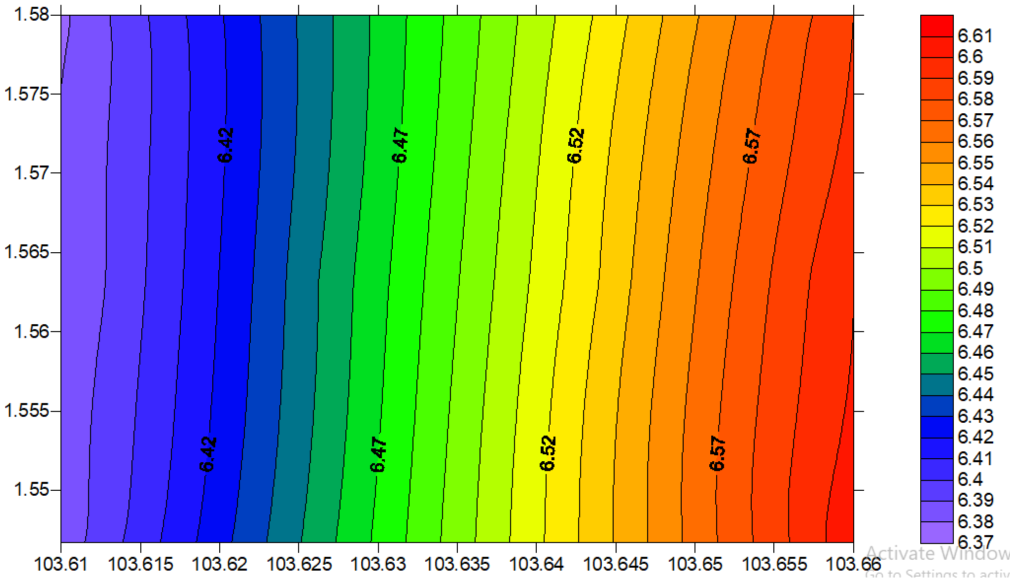


Figure 8. Gravimetric geoid map plotted by Golden Surfer Software

GNSS Data Processing

Trimble Business Center (TBC) is used as the post-processing software to process all the RINEX data obtained from fieldwork as well as from DSMM. Table 6 shows the processing strategy applied in this study while Table 7 shows the network adjustment approach.

Table 6. Processing strategy

Item	Properties/ Value
Mode	Static
Observation Time	2 hours per session
Interval	10 seconds
Ephemerides	Precise (*,sp3)
Satellite Constellation	GPS/ GLONASS
Elevation Mask	10°
Solution	Fixed

Table 7. Network adjustment approach

Item	Properties
Constraint	Fully constrained to 3 MyRTKNet CORS
Geodetic Datum	GDM2000 (2016)
Map projection system	RSO Geocentric

The contents of project file (*.vce) for planimetry part is shown in Figure 9. Only one project file is required, and the output of this project file is latitude, longitude and ellipsoidal height of 10 geodetic markers as well as their RSO Geocentric Coordinates.

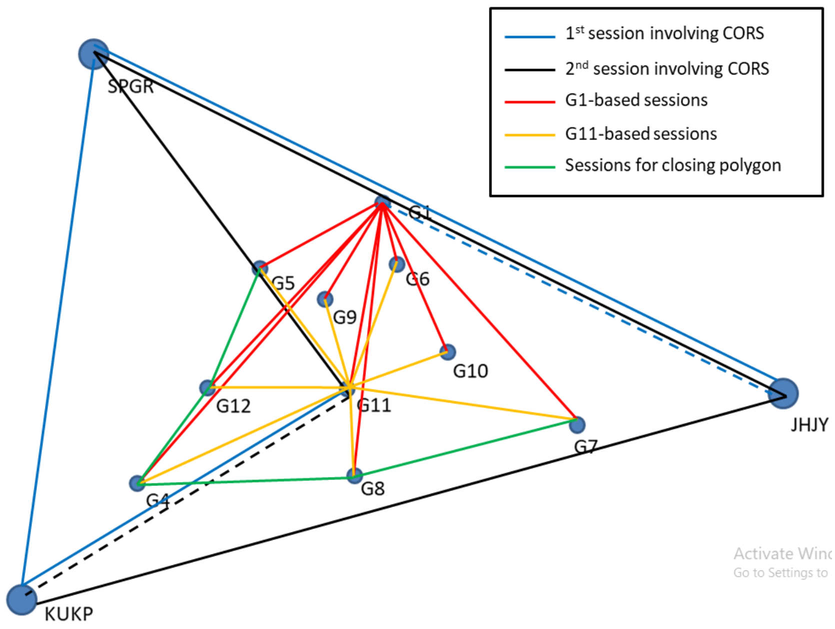


Figure 9. Planimetry project file content in this study

For the derivation of orthometric height, two different project files are created to determine the latitude, longitude and ellipsoidal height of S0394 and S1159, respectively, as shown in Figure 10.

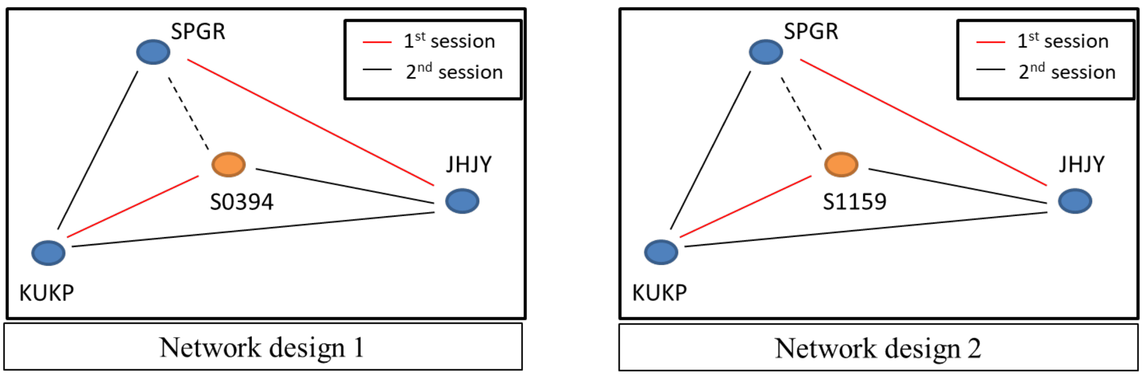
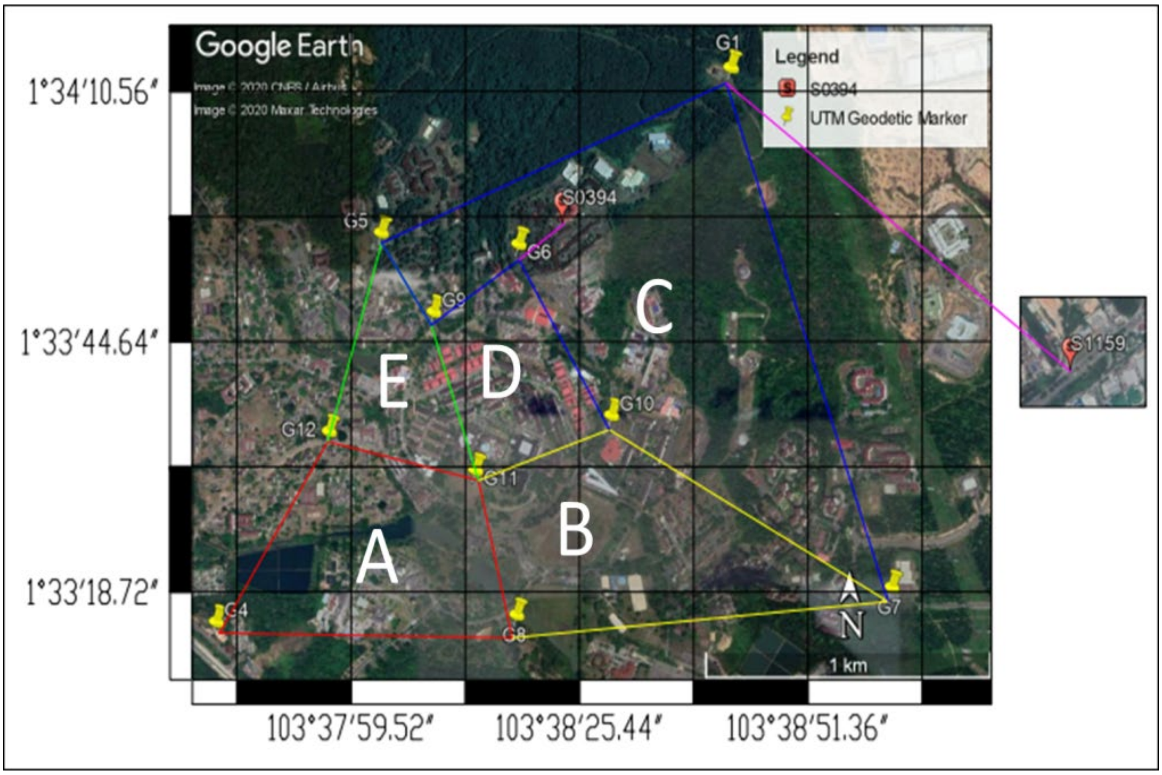


Figure 10. Two different project files are created for S0394 and S1159

1D Levelling Network Simulation

A levelling network is simulated that consists of all of the geodetic markers and SBM involved in this study as shown in Figure 11.



Source: (Modified from Google Earth, 2020)

Figure 11. Levelling network simulated in this study consisted of 5 loops and two connection lines

Based on the levelling network, difference in orthometric heights, ΔH between various geodetic markers and SBM are determined by using Equation (1).

$$\Delta H_{from\text{-}to} = h_{to} - h_{from} - (N_{to} - N_{from}) \quad (1)$$

Referring to Equation (1), h is ellipsoidal height obtained from TBC, N is a gravimetric geoid value obtained from Golden Surfer Software, and the "from" and "to" is subjected to the decision on direction of loop running, as long as returning to the same point. StarNet is used as Least Square Adjustment software to adjust the levelling network and eventually produced the adjusted and finalized orthometric height, H of all 10 geodetic markers.

Formal Error

Formal error is used as statistical analysis in this study to compare new determined coordinates with old coordinates published by MGA. The equations of difference between new and old coordinates are shown in Equation (2), (3) and (4).

$$\Delta\emptyset = (''\emptyset_{New} - ''\emptyset_{Old}) \times 30\ meters \tag{2}$$

$$\Delta\lambda = (''\lambda_{New} - ''\lambda_{Old}) \times 30\ meters \tag{3}$$

$$\Delta h = (h_{New} - h_{Old}) \tag{4}$$

Referring to Equation (2), (3) and (4), $\Delta\emptyset$, $\Delta\lambda$ and Δh is the difference in latitude, longitude and ellipsoidal height, respectively. Comparing 3D geographical coordinates is useful in quantifying the difference between geodetic datum PMGSN94 versus GDM2000 (2016)) and possible indication of dynamics of the earth. There is no such necessity to compare old MRSO Coordinate and new RSO Geocentric Coordinate because MRSO is no longer practiced (DSMM, 2009).

The value obtained from Equation (2) and (3) is used to compute two quantities of formal error: magnitude and direction. The formula of magnitude and direction is shown in Equation (5) and (6).

$$Magnitude\ = \sqrt{\Delta\emptyset^2 + \Delta\lambda^2} \tag{5}$$

$$Direction\ = tan^{-1}\left(\frac{\Delta\lambda}{\Delta\emptyset}\right) \tag{6}$$

GNSS Comparison: GPS vs GLONASS

A simple analysis is made between GPS and GLONASS. This analysis is only made on planimetry part, based on network adjustment report produced from GPS-enabled project file and GLONASS-enabled project file. Regardless of the result of comparison between GPS and GLONASS, final coordinates of UTM geodetic markers to be published in this study are based on GPS-enabled network adjustment reports, because GNSS surveys in Malaysia are still not multi-channel ready. Adding GLONASS signals to GPS does not make a noticeable improvement of coordinates' accuracy and in some cases even caused accuracy deterioration (Maciuk, 2018). Plus, some of the CORS observation files acquired from DSMM only contained GPS data without GLONASS data.

RESULTS AND DISCUSSION

Baseline Processing

Table 8 summarised the horizontal precision and vertical precision of all the 28 baselines processed in TBC.

Table 8. Baseline processing quality of all 28 baselines involved in this study

Baseline Type	Baseline	Horizontal Precision (m)	Vertical Precision (m)	Baseline ellipsoidal distance (m)
G1-based sessions	G1-G10	0.004	0.007	1168.467
	G1-G5	0.005	0.008	1318.031
	G1-G6	0.003	0.004	914.282
	G1-G9	0.010	0.019	1286.722
	G1-G7	0.005	0.007	1742.191
	G1-G4	0.004	0.021	2507.623
	G1-G11	0.002	0.003	1547.968
	G1-G8	0.003	0.005	1893.012
	G1-G12	0.004	0.007	1811.135
G11-based sessions	G11-G5	0.002	0.003	829.341
	G11-G6	0.002	0.003	740.915
	G11-G9	0.004	0.006	529.401
	G11-G10	0.002	0.004	502.378
	G11-G4	0.002	0.004	1025.565
	G11-G7	0.003	0.004	1504.503
	G11-G12	0.002	0.004	530.285
	G11-G8	0.001	0.003	491.220
Sessions of closing polygon	G12-G4	0.003	0.005	723.521
	G12-G5	0.002	0.005	664.731
	G8-G7	0.004	0.007	1319.411
	G8-G4	0.006	0.015	1041.692
CORS involving sessions	G1-KUKP	0.004	0.017	33733.863
	KUKP-SPGR	0.006	0.016	54799.116
	JHJY-SPGR	0.009	0.021	60963.782
	G11-SPGR	0.004	0.016	44895.589
	JHJY-KUKP	0.004	0.018	44321.051
	G11-KUKP	0.004	0.017	32190.052
	G1-SPGR	0.004	0.016	44832.107

All the baselines achieved a fixed solution at the first place without the need of data cleaning or data filtering, thanks to the robustness of Trimble NetR9 geodetic type receiver. Generally, baselines that involve CORS have relatively worse precision than other baselines due to its extraordinary long baseline length.

When only looking at G1-based sessions, baseline G1-G9 is the one with worst precision (10mm for horizontal and 19mm for vertical). When only looking at G11-based sessions, baseline G11-G9 which also involved marker G9 ranked as the worst baseline too (4mm for horizontal and 6mm for vertical). The reason of bad precision value on G9-involved baselines is further discussed in subsection 3.3 New Geodetic and Plane Coordinates, where the final coordinate of marker G9 is highlighted due to its highest northing and easting error.

Another marker that is worthy to highlight is marker G4. Baseline G1-G4 has bigger vertical precision (21mm) than baseline G1-G9 despite its excellent horizontal precision. However, the surrounding environment of marker G4 is good and typical as other markers and this unusually high vertical precision value is unexpected. Moreover, the final coordinate of marker G4 shows similar northing and easting error as other markers, unlike marker G9 which has highest northing and easting error. The only possible explanation regarding this phenomenon is that baseline G1-G4 is the longest baseline among all non-CORS involving baselines.

GNSS Loop Closure Test

Table 9 summarises GNSS Loop Closure Test performed right after baseline processing. 21 loops are formed with 3 legs per loop. In general, an average loop length of 30 kilometers gives a satisfactory part per million (ppm) value of 2.154.

Table 9. Summary of GNSS Loop Closure Test Result

	Length (m)	Δ3D (m)	ΔHorizontal (m)	ΔVertical (m)	PPM
Pass/ Fail Criteria					5
Best		0.003	0.003	0.000	0.047
Worst		0.038	0.017	0.037	7.428
Average Loop	30703.247	0.012	0.008	0.009	**2.154**
Standard Error	50776.018	0.015	0.009	0.012	1.867

Initially, the criteria of passing loop closure tests was set at 2 parts per million (ppm). After finding out that 9 loops failed the test, the criteria are loosened to 5 parts per million. Eventually, only one loop failed the test, with ppm value of 7.428.

The value of ppm can be computed by dividing 3D error with the total length of baselines in a loop, before times the value with 10^6. In relation to that, the shorter the baseline, the higher the resulted ppm. In this study, most of the baselines especially between UTM geodetic markers are very short, ranging from 2 kilometers to 5 kilometers, consequently forming loops with short total length of baselines. Hence, even though the value of 3D error is acceptably small, the resulting ppm value is still high enough to fail the GNSS loop closure test. For instance, the only loop that failed the test, G5-G11-G12-G5 has the value of 3D error of 15mm only, yet it failed the test due to its shortest total length of baselines in the loop, which is only 2024.623 meter.

New Geodetic and Plane Coordinates

Table 10 shows the new latitude, longitude and ellipsoidal height for all the 10 UTM geodetic markers along with the northing error, easting error and ellipsoidal height error.

Table 10. Geodetic coordinates of UTM geodetic markers in GDM2000 (2016)

Point	Latitude	Northing error (m)	Longitude	Easting error (m)	Ellipsoidal height (m)	Ellipsoidal height error (m)
G1	1°34’10.83459”	0.004	103°38’42.14884”	0.005	142.836	0.022
G4	1°33’13.74652”	0.005	103°37’44.15515”	0.006	48.049	0.023
G5	1°33’54.40667”	0.005	103°38’02.75721”	0.005	47.713	0.023
G6	1°33’53.19968”	0.005	103°38’18.32059”	0.006	53.511	0.023
G7	1°33’17.18287”	0.005	103°39’00.43678”	0.006	22.836	0.024
G8	1°33’14.26203”	0.005	103°38’17.85114”	0.005	22.472	0.023
G9	1°33’46.13750”	0.007	103°38’08.52524”	0.009	30.286	0.027
G10	1°33’35.32872”	0.005	103°38’28.57793”	0.006	25.383	0.024
G11	1°33’29.60112”	0.004	103°38’13.35446”	0.005	42.011	0.022
G12	1°33’33.64798”	0.005	103°37’56.67714”	0.005	33.179	0.023

Table 11 and 12 list the new plane coordinates of all the 10 UTM geodetic markers in RSO Geocentric and Cassini-Soldner Geocentric, respectively.

Table 11. RSO Geocentric Coordinates of UTM geodetic markers

Point	Easting (meter)	Northing (meter)
G1	627613.693	173587.843
G4	625820.450	171835.506
G5	626395.997	173083.931
G6	626876.984	173046.623
G7	628178.094	171939.870
G8	626861.881	171850.818
G9	626574.138	172829.888
G10	627193.728	172497.632
G11	626723.139	172321.965
G12	626207.766	172446.505

Table 12. Cassini-Soldner Geocentric Coordinates of UTM geodetic markers with origin set at Johor state

Point	Easting (meter)	Northing (meter)
G1	9344.617	-52288.681
G4	7552.057	-54042.225
G5	8127.018	-52793.316
G6	8608.093	-52830.372
G7	9909.978	-53936.589
G8	8593.626	-54026.355
G9	8305.320	-53047.300
G10	8925.175	-53379.272
G11	8454.613	-53555.215
G12	7939.101	-53430.932

Overall vertical errors are higher than horizontal errors, as Mohamed, Doma and Rabah (2019) and Hofmann-Wellenhof, Lichtenegger and Wasle (2008) explained that GNSS receivers are not able to track satellites below the horizon, consequently horizontal precision are better than vertical precision. G9 as the marker with both highest horizontal error and vertical error (Northing error = 9mm, easting error = 7mm, ellipsoidal height error = 27mm), which can be explained by the relatively heavier canopy around marker G9 as compared to other markers. Braasch (2007), Konnestad (2018) and Ramachandran *et al.* (2019) suggested that accuracy of GNSS measurements is affected by nearby conditions. In particular, canopies turned out to be a huge obstacle for GNSS carrier-phase measurement. Figure 12 shows the surrounding environment of marker G9 during observation

Markers G1 and G11 both have the lowest northing error (4mm) and lowest vertical error (22mm) as they act as base markers and occupy for very long duration as compared to other markers, thanks to the open sky environment, which is suited for static observation.

Figure 12. Surrounding environment of marker G9 during observation in North (left top), East (right top), South (left bottom) and West (right bottom)

Orthometric Height in NGVD

Table 13 summarises the final adjusted orthometric heights of UTM geodetic markers which is consistent with NGVD. The final adjusted orthometric heights of S0394 and S1159 are also included in Table 13.

Table 13. Final adjusted orthometric height of UTM geodetic markers

Point	Orthometric Height (m)	Standard Deviation (m)
G1	134.749	0.003
G4	40.028	0.003
G5	39.672	0.002
G6	45.451	0.001
G7	14.720	0.003
G8	14.409	0.002
G9	22.237	0.002
G10	17.308	0.002
G11	33.955	0.002
G12	25.144	0.002
S0394	62.215	Fixed
S1159	34.960	0.010

After several approaches, final adjusted orthometric heights in Table 13 are based on the decision to fix S0394 but unfix S1159 during Least Square Adjustment using StarNet software. This is because the route G1-S1159 has the highest standard error (10mm) due to the longest distance (11.031 km) among all the simulated routes in the levelling network. Hence, the decision of unfixing S1159, which is located about 11 kilometers away from the campus area, can minimize the distance-dependent error that propagated into the final adjusted orthometric height of markers, as the effectiveness of double-differencing is limited when dealing with long baselines and residuals due to distance-dependent errors will remain (Musa *et al.*, 2006).

It is worthy to mention that the decision to unfix S1159 is supported by a result of trial-and-error levelling network adjustment, which is by fixing only marker G1 (known value taken from adjustment result of fixing both SBM). Table 14 shows the adjusted orthometric height of both SBM from this trial-and-error adjustment and their difference from DSMM published value.

Table 14. Adjusted orthometric height of SBM versus DSMM published value

Point	Orthometric Height (m)	Standard Deviation (m)	DSMM Published Orthometric Height (m)	Difference (m)
G1	134.740	Fixed	-	-
S0394	62.206	0.003	62.215	-0.009
S1159	34.951	0.010	34.875	0.076

Since this is only trial-and-error adjustment, the orthometric heights of the remaining geodetic markers are not shown to prevent readers' confusion with Table 13. Anyway, this attempt proves that S1159 is the one that carried a large error value because its adjusted orthometric height differs much from DSMM published value (76mm) as compared to S0394 which is only 9mm difference from DSMM published value.

Interestingly, by referring back to Table 13, the deviation of S1159's adjusted orthometric height from DSMM published value is 85mm, which is the absolute summation of difference in Table 14 (85mm = 76mm + 9mm). This can be interpreted as: the decision of fixing S0394 and unfixing S1159 has resulted in the error of S0394 (9mm) being propagated into the final adjusted orthometric heights in this study while preventing S1159's error (76mm) from doing the same thing.

This method of showing stability of SBM is not true and only trial-and-error. In fact, a third SBM should be used for this purpose, and not a geodetic marker that does not have known orthometric height.

Formal Error

Table 15 summarises the formal error between old geodetic coordinates published by MGA and new geodetic coordinates determined by this study. The differences in ellipsoidal height are also included.

Table 15. Formal Error: PMGSN94 versus GDM2000 (2016)

Point	$\Delta\emptyset$ (m)	$\Delta\lambda$ (m)	Magnitude (m)	Bearing	Δh (m)
G1	-1.313	0.333	1.355	165°45’26.30”	0.5622
G4	-1.309	0.340	1.352	165°26’07.02”	0.5833
G5	-1.305	0.337	1.348	165°32’28.80”	0.5871
G6	-1.310	0.337	1.353	165°34’29.50”	0.5538
G7	-1.315	0.326	1.354	166°05’36.01”	0.5357
G8	-1.311	0.335	1.353	165°41’08.00”	0.5716
G9	-1.302	0.353	1.349	164°49’03.82”	0.5407
G10	-1.304	0.346	1.349	165°07’53.87”	0.5960
G11	-1.312	0.336	1.354	165°38’44.70”	0.5669
G12	-1.306	0.331	1.347	165°47’28.90”	0.5407

The differences in Table 15 are similar to the differences concluded by Jaffar, Musa and Aris (2019) in which overall displacement of MyRTKNet stations are 34.6cm in the direction of 111.1 degree from GDM2000 (2009) in ITRF2000 to the ITRF2014 at epoch 2016. The bigger average magnitude in this study can be explained by the bigger time differences between PMGSN94 and GDM2000 (2016) as compared to that between GDM2000 (2009) and ITRF2014. Nevertheless, this study proved that UTM geodetic markers have displaced as the same things happened on MyRTKNet (DSMM, 2009b; Yazid *et al.*, 2019). Coordinate revision is absolutely required. Figure 13 visualises the formal error between PMGSN94 and GDM2000 (2016).

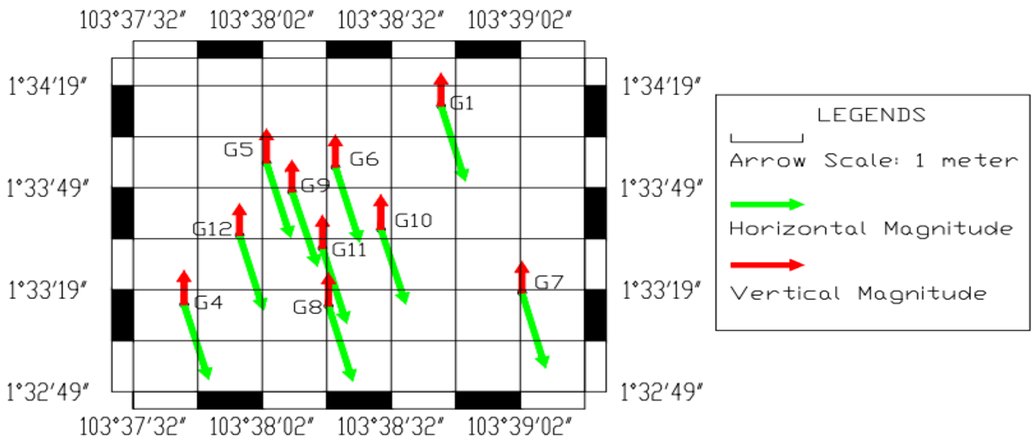


Figure 13. Vector plot map that visualizes the difference between new coordinate and old coordinate

GPS vs GLONASS

Table 16 compares the easting error, northing error and ellipsoidal height error of all UTM geodetic markers, which were obtained from two different GNSS-enabled project files: GPS and GLONASS.

Table 16. Comparison between GPS and GLONASS

Point	Easting error (m)		Northing error (m)		Ellipsoidal height error (m)	
	GPS	GLONASS	GPS	GLONASS	GPS	GLONASS
G1	0.005	0.007	0.004	0.006	0.022	0.028
G4	0.006	0.014	0.005	0.011	0.023	0.036
G5	0.005	0.009	0.005	0.009	0.023	0.032
G6	0.006	0.009	0.005	0.007	0.023	0.030
G7	0.006	0.021	0.005	0.013	0.024	0.037
G8	0.005	0.013	0.005	0.010	0.023	0.034
G9	0.009	0.037	0.007	0.026	0.027	0.070
G10	0.006	0.013	0.005	0.011	0.024	0.032
G11	0.005	0.006	0.004	0.006	0.022	0.028
G12	0.005	0.013	0.005	0.009	0.023	0.035

It is obvious that the performance of the GPS constellation is better than the GLONASS constellation in this study. The result of comparison is consistent with the study carried out by Ristanto, Khomsin and Anjasmara (2019) in which the use of GLONASS satellites alone has provided the lowest precision of all tactics. GPS is more popular because it delivers higher precision signals compared to GLONASS in a wide variety of countries around the world despite the fact that GLONASS functions more effectively at northern latitude, as GLONASS was designed to operate in Russia (Abdul Majeed, 2017). Nevertheless, Geng and Shi (2016) and Liu *et al.* (2017) presented various approaches of combining GPS and GLONASS in resolving ambiguity for precise point positioning (PPP) and have obviously proven to reduce the initialization periods.

CONCLUSION

This study provides newly redefined geodetic and plane coordinates on UTM geodetic markers by using geodetic type receivers for the purpose of coordinate revision. A brand-new set of geodetic coordinates which are tied to GDM2000 (2016) geodetic datum are produced to UTM geodetic markers. Two sets of plane coordinates, RSO Geocentric and Cassini-Soldner Geocentric, are determined, plus geodetic markers can now serve for both mapping and cadastral purpose. Moreover, UTM geodetic markers are assigned with new orthometric heights which are consistent with the NGVD, providing accurate vertical reference for levelling work.

Geomatics students can now utilize these geodetic infrastructures for their practical work and research work with full assurance. Additionally, UTM geodetic markers can now serve cadastral and engineering purposes especially by industrial practitioners around Johor as Cassini-Soldner Geocentric and RSO Geocentric coordinates are produced in this study. The method of establishing 3D GNSS control network in this study are recommended for application in high-precision survey work such as in establishing Ground Control Point (GCP) for aerial photogrammetry, engineering survey, cadastral and deformation projects.

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