Realization of New Korean Horizontal Geodetic Datum: GPS Observation and Network Adjustment

Young-Jin Lee[1], *Hung-Kyu Lee[2], Gwang-Ho Jung[1],

[1]Department of Construction & Geoinformatics Engineering
Kyungil University. Gyeongbuk, Republic of Korea (E-mail: yjlee@kiu.ac.kr)
[2] Department of Civil Engineering
Changwon National University, Gyeongnam, Republic of Korea (E-mail: hkyulee@changwon.ac.kr)

Abstract

New geocentric geodetic datum has recently been realized in Korea, Korean Geodetic Datum 2002- KGD2002, to overcome problems due to the existing Tokyo datum, which had been used in this country since early 20th century. This transition will support modern surveying techniques, such as Global Navigation Satellite Systems (GNSS) and ensures that spatial data is compatible with other international systems. For this realization, very long baseline interferometry (VLBI) observations were initially carried out in 1995 to determine the coordinates of the origin of KGD2002 based on the International Terrestrial Reference Frame (ITRF). Continuous GPS observations were collected from 14 reference stations across Korea to compute the coordinates of 1st order horizontal geodetic control points. During the campaign, GPS observations were also collected at about 9,000 existing geodetic control points. In 2006, network adjustment with all data obtained using GPS and EDM since 1975 has been performed under the condition of fixing the coordinates of GPS continuous observation stations to compute coordinate measurements of the 2nd and 3rd geodetic control points. This paper describes the GPS campaigns which have been undertaken since 1996 and details of the network adjustment schemes. This is followed by presenting preliminary results of the 2nd-order GPS network adjustment.

Keywords: Geodetic Datum, KGD2002, GPS, Network Adjustment

1. Introduction

A datum is defined as any numerical or geometrical quantity or set of quantities which serve as a reference or base for other quantities[1]. In geodetic surveying, two types of datums are considered: a horizontal datum which forms the basis for the computation of horizontal control survey that consider the curvature of the earth, and a vertical datum to which elevations are referenced. The horizontal datum is traditionally realized by determining the geodetic position (e.g., latitude and longitude) of an origin point, an azimuth of a line to another geodetic control point, and the geoid separation at the origin point with defining the parameters (e.g., the equatorial radius and the flattening) of the ellipsoid, which should be a best fit to the geoid over a whole country. A geodetic network extends the datum across a nation for various applications of surveying and mapping. Geodetic surveying techniques, such as triangulation, trilateration, and traverse have traditionally been used to determine the coordinates of the control points within the network. In addition, network adjustment has played an important role in reducing the effect of observational errors in the coordinate estimation to a minimum[2)3)]. Considering the abovementioned procedure, it should be noted that a change in the datum parameters affects every point on the geodetic network.

Geodetic surveying to establish the triangulation network began in Korea in 1910 and was conducted by the Bureau of Land Survey with the cooperation of the Japanese Military Land Survey. During the project, 34,444 geodetic control points were established along the Korean Peninsula. Among them, 16,089 were situated in South Korea. However, it is important to note that with triangulation, establishment of the network was accomplished by connecting with the Japanese geodetic network, instead of defining a national geodetic datum. This means that the Tokyo datum has been adopted in Korea. It is well known that the technique used in the Tokyo datum realization was single astronomic point datum orientation, in which the geoid and ellipsoid were assumed to be the same at the origin point. This technique is the simplest means of establishing the datum, and as a result large systematic errors may be introduced in the geodetic network as the survey is expanded. Hence, it is obviously inappropriate to use the Tokyo datum in the geodetic survey in Korea. Nevertheless, the Tokyo datum has been used in Korea for a century. Although there was an attempt in the mid 1980s to establish a Korean datum in the mid of 1980's by the astronomical geodetic orientation technique which considers the deflection of the vertical at a number of Laplace stations including the datum origin, it could not be connected with the existing geodetic network. This was mainly because the advent of new space geodetic surveying techniques (e.g., GNSS, SLR, VLBI) caused the Korean government to change its plan to establish the global geocentric datum, which is a best fit to the geoid over the entire earth.

Korea has adopted a new geocentric datum, the Korean Geodetic Datum 2002- KGD2002, to replace the existing Tokyo datum. This is semi-dynamic datum with coordinates aligned to the International Terrestrial Reference Frame 2000 (ITRF 2000), according to the reference data of 1st January 2002.0 (Epoch2002.0). This was achieved by determining the coordinates of the datum origin and the first-order geodetic control points, which consist of 14 GPS Continuous Operating Stations (CORS), through very long baseline interferometry (VLBI) and GPS observations. This datum transition will support modern surveying techniques, such as Global Navigation Satellite Systems (GNSS), and will ensure that spatial data is compatible with other international systems. The network adjustment of the 2^{nd} and 3^{rd} order geodetic control points has been undertaken to connect them to the KGD2002. The adjustment includes approximately 25,000 points observed by either EDM or GPS since 1975.

Since 1996, the National Geographical Information Institute (NGIIS) of Korea has carried out GPS campaigns not only for the determination of the datum transition parameters but also for maintaining and upgrading the geodetic network. In this paper, the ongoing GPS network adjustment in Korea is discussed first, followed by a brief description of GPS observations and their baseline processing. Finally, preliminary results of the 2^{nd} order GPS network with emphasis on the stochastic modeling scheme and general accuracy over the adjustment will be presented.

2. The Network Adjustment Procedure

It is standard that an adjustment for large scale geodetic network begins with arranging and checking all of the observations. In GPS network adjustment, it is most important to check antenna type and height, and to confirm the observation point names. Either defining the inappropriate antenna type or entering the inaccurate antenna height introduces a systematic error in the vertical estimation. In addition, inconsistencies in reoccupied points result in computer process errors during network adjustment. All the GPS data used in this project (i.e., about 10,000 points) have been closely examined.

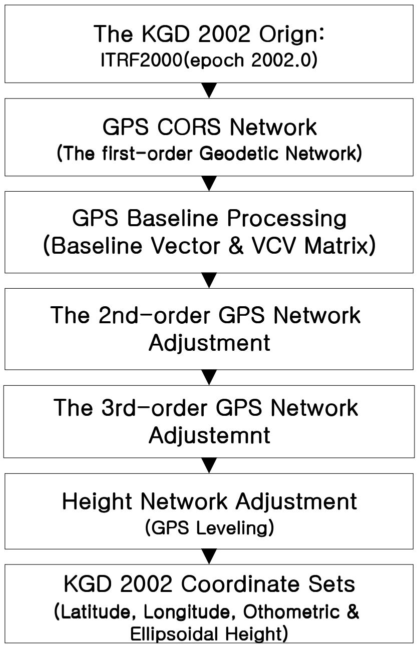

Figure 1. General procedure for the GPS network adjustment

The Korean geodetic network comprises three hierarchical levels, such as the 1^{st}, 2^{nd} and 3^{rd} order control points. Since the determination for the coordinates of the 1^{st} order network, which consists of 14 GPS CORSs was made during the KGD2002 realization, the main purpose of this project is to adjust the 2^{nd} and the 3^{rd} order networks so as to determine the KGD2002 coordinates. Details on the adjustment procedure are illustrated in Figure 1. In such an adjustment, every lower level network will be connected to the higher level network by at least three well distribute points. This ensures that the network being attached can not rotate relative to the higher level network. Hence, the 2^{nd} order nationwide network will be simultaneously adjusted under the condition that three dimensional coordinates of all the 1^{st} order control points are fixed. On the other hand, since the 3^{rd} order network consists of eight block networks comprising almost 10,000 points, it is planned that each of the block networks be separately adjusted by fixing the coordinates resulting from the 2^{nd} order network adjustment. Finally, the height network will be adjusted with respect to benchmarks which are connected to the 3^{rd} order network by GPS observations, so as to determine the orthometric heights of the horizontal control points. Hence, the outcomes of the three different level adjustments will be geographical coordinates (i.e., latitude and longitude), ellipsoidal heights, and orthometric heights of all of the control points, the so-called the KGD2002 coordinates.

3. GPS Observation and Processing

3.1 GPS observations

The National Geographical Information Institute (NGII) of Korea, with cooperation of a number of surveying contractors, has held GPS observation campaign over the geodetic network since 1996. During these campaigns, about 10,000 points were observed until the end of 2005. The Korean specification for GPS control surveying was applied so that the campaigns could achieve high levels of surveying efficiency and accuracy.

Table 1 shows a summary of GPS data which will be used in this network adjustment. The main differences between the two networks are baseline length and GPS receiver occupation time. As given in Table 1, while the baseline lengths of the 2^{nd} order network range from 20km to 120km, those of the 3^{rd} order network do not exceed 5km. GPS occupation times were eight hours for the 2^{nd} and four hours for the 3^{rd} order networks, respectively.

Table 1. Summary of GPS data used in the adjustment

Order	Baseline Length	Recoding Time	Num. of Stations	Num. of Campaigns
2^{nd} order Network	40~120km	8 hours	200	8
3^{rd} order network	2~5km	4 hours	8,744	66

3.2 GPS baseline processing

All the GPS data was initially processed by the surveying contractors, who made filed observations. Therefore, a variety of different commercial software was used. However, in the case of GPS short baseline up to 10km, processing results are almost identical no matter which software is used. This is because applying the double-differencing technique to the GPS observation can significantly reduce or even eliminate common error sources between a reference and a rover receiver. Hence, for this project, the processed baseline vectors, together with their variance-covariance (VCV) matrices, will be used in the subsequent adjustment unless significant problems (e.g., incorrect antenna heights and/or reference coordinates) exist.

The baseline length of the 2^{nd} order network ranges from a few tens of km to a maximum of 120 km, which corresponds to GPS medium baseline in standpoint from data processing. In this case, it is possible that the processing results (e.g., baseline vector and

VCV matrix) were different depending on the software used. This is mainly due to the fact that each of the commercial GPS software has slightly different functional and stochastic modeling schemes, especially for handling baseline length dependent errors (e.g., atmospheric effects). Because of this, all the GPS data was reprocessed using the Leica Geomatics Office (LGO) static processing module in order to ensure consistent results. Each campaign was manually processed with the nearest GPS CORS. Options to reduce the distance dependent errors include applying linear combination methods and precise ephemeris provided by International GNSS Services (IGS). The reprocessing provided all of the final baseline solutions with ambiguity-fixed. In order to ensure the solution quality, every loop closure was checked according to the specification for GPS control surveying.

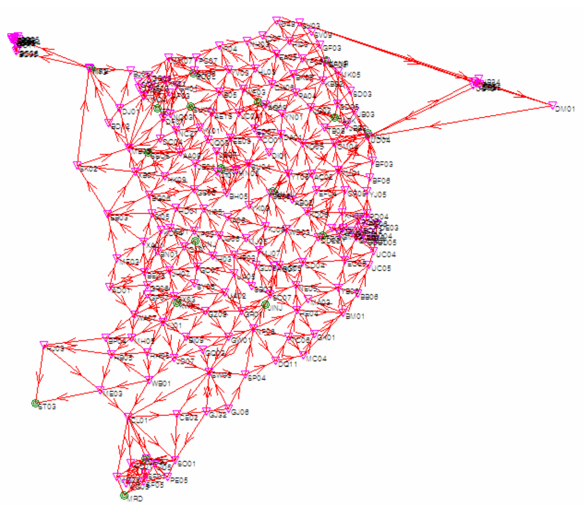

Figure 2. Reprocessed 2nd order GPS network

4. The 2nd-Order Network Adjustment

4.1 The adjustment strategy

There are essentially two classes of network adjustment for geodetic surveying: *'minimally constrained adjustment'* and *'over constrained adjustment.'* The former assumes that in the case of a GPS network, only one station is held fixed, meaning that the coordinates are not permitted to move (or adjust). This is required to avoid the normal equations becoming singular. The minimally constrained adjustment is mainly performed to validate a measurement, check for outlier existence, and look into the internal consistency of the measurement, namely the precision of the derived coordinates. In this project, three different levels of the minimally constrained adjustment were carried out in order to rigorously examine outliers and modify the stochastic model (i.e., VCV matrix) resulting from the GPS baseline processing. The first two processes, *'campaign adjustment'* and *'integration adjustment,'* focus on detecting outliers by performing the Tau test against standardized residuals resulting from the adjustment. In these adjustments, the VCV matrices are scaled through multiplication by the posterior variance factor. However, this stochastic modeling scheme only takes internal errors into account. After successful adjustment (i.e., all outliers are removed), an empirical stochastic modeling scheme to consider both the internal and external errors is applied. This is because the VCV matrix resulting from the GPS processing is likely to be over-optimistic. Therefore, an iterative process is applied for the modeling until the χ^2 test is passed. Note that results of the minimally constrained adjustment generally indicate the precision of the derived coordinates.

The over constrained adjustment is carried out by fixing at least three stations in order to define the datum, orientation, and scale of the network. The adjustment should not be performed until all obvious outliers have been detected and removed or remeasured. In this research, all available GPS CORS were held fixed in the over constrained adjustment.

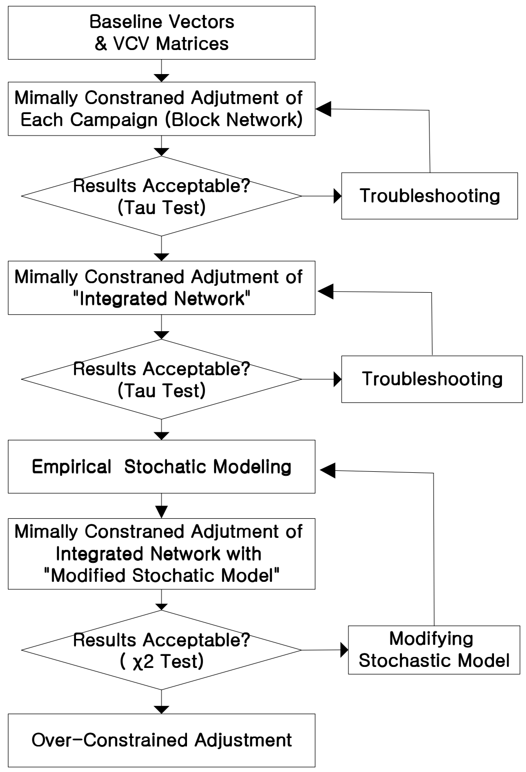

Figure 3. The 2nd order network adjustment flow

4.2 Result of the minimally constrained adjustment

Campaign adjustments were performed eight times by fixing the nearest GPS CORS. As shown in Table 2, 21 baselines were identified as outlier from the standardized residual (Tau) test, so that they were removed. After merging all these campaigns into a single network (i.e., integration network), the second adjustment was carried out, so that additional five baselines were identified as outlier (see, the values in the parenthesis in Table 2).

Table 2. The number of baselines identified as outliers

Campaign	1997	1998	1999	2000-1
Outliers	5	4	2(3)	2
Campaign	2000-2	2000-S	2000-W	2000-J
Outliers	2	1	5(2)	0

An empirical method was applied to determine realistic variances for GPS baseline solutions[3]. This was performed by assigning an absolute constant value and baseline length dependent value. It was possible to derive these values from several iterative processes until the variance factor test (χ^2) was passed: 5mm and 0.5PPM according to the baseline component in the Cartesian coordinate system, which corresponds to 4mm+0.4PPM for the horizontal component and 8mm+0.8PPM for vertical the component in the topocentric form. All the VCV matrices were constructed by using the variance values represented in the topocentric coordinate system and the coefficient of correlation of the original VCV matrices. Subsequently, each of the campaign networks was readjusted. On examination of internal consistency of the campaign networks, a comparison of 31 common points among the networks was made and given in Table 3. These results indicate that that the campaign networks agree upon around 3.0cm in the horizontal and around 5cm in the vertical component, respectively.

Table 3. Statistical summary of coordinate differences of the common points among the campaign networks

Component	Mean	RMS	Max.	Min.
Horizontal(2D)	0.014m	0.014m	0.034m	0.0m
Vertical(1D)	0.032	0.020m	0.075m	0.004m

A minimally constrained adjustment of the integration network was finally performed by applying the modified stochastic model (e.g., the VCV matrix). Its results are summarized in Table 5. A posterior variance factor 1.04 passed the Chi-square test, ensuring the fidelity of both the functional and the stochastic models used in the adjustment. Figure 4 illustrates a histogram of baseline vectors (i.e., observations), clearly showing no outliers in the observations used.

Table 4. Summary of the minimally constrained adjustment

Fixed Point in 3D	SUWN
Number of Station	214
Coordinate Parameters	639
Number of Observations	2313
Degree of Freedom	1674
Variance Used	Horizontal:4mm+0.4PPM Vertical : 8mm+0.8PPM
Posterior Variance Factor	1.04
Chi-Square Test	Passed

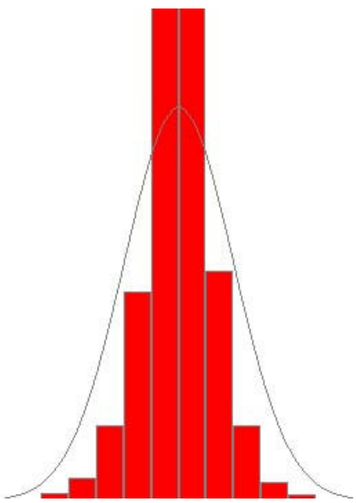

Figure 4. Histogram of the baseline vector residuals resulting from the minimally constrained adjustment

A comparison of the adjusted coordinates with the existing 1st order geodetic control points (i.e., GPS CORSs) was made (see, e.g. Table 5) in order to look over the errors associated with each coordinate. From these results, it would be possible that the general accuracy of the network adjustment is about 2.5cm in the horizontal and 4.5cm in the vertical component, respectively.

Table 5. Summary of difference between the adjusted and existing coordinates of GPS CORS

Component	Mean	RMS	Max.	Min.
Horizontal	0.012m	0.011m	0.020m	0.0m
Vertical	0.026	0.020m	0.070m	0.004m

Relative confidence with respect to 95% probability was computed from the VCV matrix of the estimated parameters (see, e.g., Table 6). It is of interest that the results are very similar with those given in Table 3 and Table 5. This is a kind of evidence that the modified stochastic model adopted in the final adjustment realistically considered the observation errors used.

Table 6. Summary of relative confidence regions (95%) resulting from the minimally construed adjustment

Component	Mean	RMS	Max.	Min.
Horizontal(2D)	0.016m	0.008m	0.081m	0.005m
Vertical(1D)	0.036	0.017m	0.176m	0.011m

4.3 Result of the over constrained adjustment

As the final stage of the 2nd order network adjustment determining the KGD2002 coordinates, an over constrained adjustment was carried out with the 1st order geodetic control points (i.e., GPS CORSs) held fixed. As shown in Table 7, total 603 coordinate parameters (e.g., latitude, longitude, and ellipsoidal height at 201 control stations) were estimated from the adjustment by making use of 2,313 observations. A Chi-test was passed with a posterior variance value of 1.06 through applying the stochastic model identical with one deriving from the previous minimally constrained adjustment. In order to ensure the existence of any outlier not being removed before the adjustment, a histogram of total baseline vectors (i.e., observations) is given in Figure 5, showing no evidence of an outlier.

Table 7. Summary of the over constrained adjustment

Fixed Point in 3D	1st-order control stations (13)
Number of Station	214
Coordinate Parameters	603
Number of Observations	2313
Degree of Freedom	1710
Variance Used	Horizontal:4mm+0.4PPM Vertical : 8mm+0.8PPM
Posterior Variance Factor	1.06
Chi-Square Test	Passed

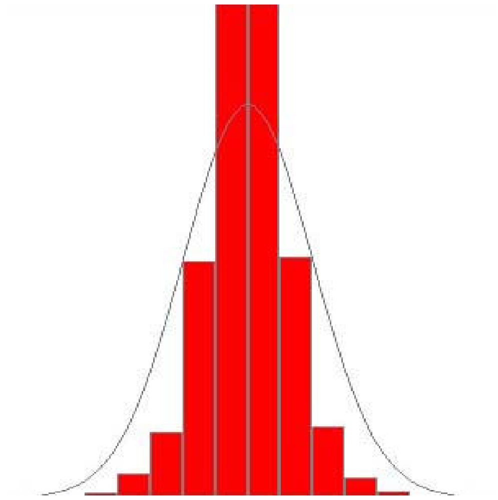

Figure 5. Histogram of the baseline vector residuals resulting from the over constrained adjustment

Relative confidence regions (95%) were computed from the VCV matrix of the estimated parameter. These results are almost identical with those deriving from the minimally constrained adjustment. Hence, it is implied from the results that the coordinates held fixed in the adjustment were more precise than the observations.

Table 8. Summary of relative confidence regions (95%) resulting from the over constrained adjustment

Component	Mean	RMS	Max.	Min.
Horizontal(2D)	0.017m	0.008m	0.086m	0.005m
Vertical(1D)	0.036	0.018m	0.185m	0.012m

Upon examination of the accuracy of the estimated coordinate resulting from this adjustment, absolute confidence regions with respect to 95% probability were computed for the horizontal and vertical coordinate component and summarized in Table 6. In addition, absolute confidence ellipsoids and bars (95%) with the adjusted network drawing are illustrated in Figure 9. the average size of the ellipsoids and bars is about 2cm and 5cm, respectively. However, both of the RMS and maximum values are relatively large. This is mainly caused by control stations situated in the offshore islands about 150km from the seashore. In order to determine point estimation accuracy of the onshore and inshore networks, the statistics of the absolute confidence regions were recomputed by excluding the 23 points of the offshore islands (see, Table 6). It is clear that the RMS and maximum values become much smaller compared with those in Table 8. Hence, it is possible to conclude that the average accuracy of the estimated coordinates within the onshore network is better than 2cm and 4cm in the horizontal and vertical component, respectively.

Table 9. Summary of absolute confidence regions (95%)

Component	Mean	RMS	Max.	Min.
Horizontal(2D)	0.021m	0.013m	0.084m	0.008m
Vertical(1D)	0.047	0.030m	0.182m	0.018m

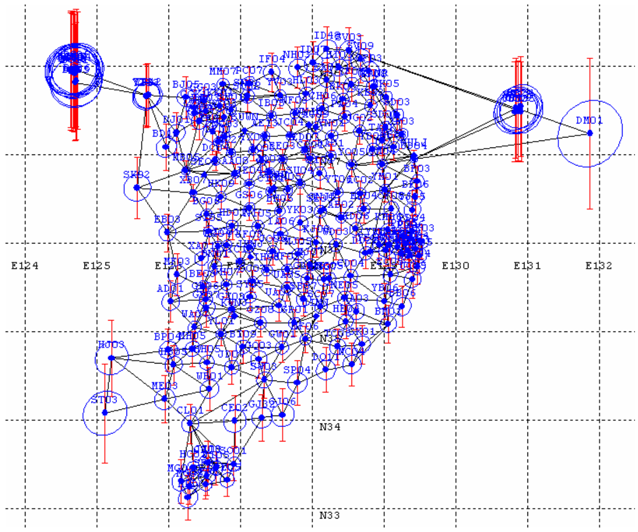

Figure 6. Adjusted network with absolute confidence regions

Table 10. Summary of absolute confidence regions excluding points in offshore islands (95%)

Component	Mean	RMS	Max.	Min.
Horizontal(2D)	0.017m	0.005m	0.034m	0.008m
Vertical(1D)	0.038	0.011m	0.076m	0.018m

5. Concluding Remarks

Upon examination of the accuracy of the estimated coordinate resulting from this adjustment, absolute confidence regions with respect to 95% probability were computed for the horizontal and vertical coordinate component and summarized in Table 6. In addition, absolute confidence ellipsoids and bars (95%) with the adjusted network drawing are illustrated in Figure 9. the average size of the ellipsoids and bars is about 2cm and 5cm, respectively. However, both of the RMS and maximum values are relatively large. This is mainly caused by control stations situated in the offshore islands about 150km from the seashore. In order to determine point estimation accuracy of the onshore and inshore networks, the statistics of the absolute confidence regions were recomputed by excluding the 23 points of the offshore islands (see, Table 6). It is clear that the RMS and maximum values become much smaller compared with those in Table 8. Hence, it is possible to conclude that the average accuracy of the estimated coordinates within the onshore network is better than 2cm and 4cm in the horizontal and vertical component, respectively.

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