Validating VRS-based Post-processing Kinematics GPS for Horizontal Densified Surveying

Ming Da Tsai[1], **Chia Lung Wu**[2], **Yu Ching Lin**[1] **and Ming Chih Wang**[3]

[1] *Department of Environmental Information and Engineering, CCIT, National Defense University.*
[2] *National Land Surveying and Mapping Center, Taiwan.*
[3] *Department of History and Geography, University of Taipei, Taipei City, Taiwan.*

Abstract

A virtual station-based post-processing kinematic (VRS-PPK) positioning technique is advantageous for operations that do not require users to establish a base station or employ a real-time communication device. Through a VRS system, this technique is potentially applicable for horizontal densified surveying. This study adopted the static observation data obtained through the control station resurvey in 2012 (conducted by the Taiwan Ministry of the Interior) to examine the internal precision and external accuracy of VRS-PPK positioning, thereby providing application standards for using this technique to conduct horizontal densified surveying or other types of surveying. The results revealed that after the testing sites were screened through the criterion that the standard deviation of each observation set must be less than 2 cm in horizontal component and less than 5 cm in height component, the success rate of testing site positioning was determined to be 70%. The morning session attained the highest success rate, and the average standard deviation of the plane coordinates was 1.3 cm. Next, the difference between the two observation sets of each testing site coordinate was examined. The results revealed that 92% of the testing sites attained a set difference not exceeding 2 cm in either N or E component. However, when the tolerance for set difference was increased to 3 cm, 97% of the testing sites could meet the set difference standard. Subsequently, the baseline length differences between the VRS-PPK and static solutions were examined. The results indicate that the specification for examining baseline length differences can be tightened up to 20 mm + 4 ppm × L, which can serve as a standard for control station resurvey using VRS-PPK.

Keywords: VRS, RTK, PPK, Horizontal Densified Surveying

虛擬基準站後處理 GPS 動態定位進行平面加密測量之適用性探討

蔡明達[1*] 吳嘉隆[2] 林玉菁[1] 王明志[3]

[1] 國防大學理工學院環境資訊及工程學系

[2] 內政部國土測繪中心

[3] 台北大學歷史地理系

摘　　要

VBS-PPK 定位解算技術具有不須自行設置基站，也不須配置即時通訊裝置之條件，在配合 VBS 系統之運作下，具有進行平面加密測量之潛在優勢。本研究利用 2012 年內政部辦理基本控制點檢測工作之靜態觀測資料，配合觀測數據所產製之 VBS 觀測量，進行 VBS-PPK 定位成果之內、外部精度(interior and external precision)檢視，以提供該技術辦理平面加密測量或辦理檢測之適用標準。由研究成果可知，採用平面 2 cm 及高程 5 cm 進行測回(set)中誤差(standard deviation)之初步篩選後，觀測組之定位解算成功率僅達 70%，且以上午時段之成功率為最佳，整體資料之平面坐標中誤差為 1.3 cm。再針對二組測回平均坐標(coordinate)求取較差(set difference)時，任一平面分量(horizontal component)超出 2 cm 標準之點位數量可達 8%，若可放寬至較差小於 3 cm 時，測點之通過率將可達到 97%。通過中誤差及測回較差檢核之 VBS-PPK 定位成果，在組成基線(baseline)進行其標準長度之差異檢核時，其成果反映之檢核標準可較參考規範(specification)拉高至 20 mm+4 ppm × L，以作為 VBS-PPK 辦理平面控制點檢測(re-survey)標準使用。

關鍵詞：虛擬基準站，即時動態定位，後處理動態定位，平面加密測量

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Manuscript received January 25, 2019; revised November 22, 2019;* Corresponding author

I. INTRODUCTION

In recent years, rapid development in internet and wireless communication techniques has facilitated extending global positioning system (GPS) technology from conventional static observations to real-time kinematic (RTK) operations that involve wireless data transmission. Furthermore, virtual reference station-based real time kinematic (VRS-RTK) has been developed to enable real-time positioning through integrating wireless communication with network operations of GPS reference stations.

Because Taiwan is located in the region where interactions between the Eurasia and Philippine plates occur actively, frequent crustal movement causes continual and noticeable changes in the control stations. Therefore, establishing a continuously operating GPS reference network to maintain the accuracy of the national geodetic system and providing real-time and high-precision positioning services are imperative missions of the National Land Surveying and Mapping Centre (NLSC) of Taiwan. Since 2004, the NLSC has planned to establish a nationwide e-GPS reference network with a VRS-RTK system, in which the Trimble GPSNet system is adopted to calculate regional correction parameters (Wang et al., 2006). This enables increasing not only the effective operating range but also positioning precision and reliability of RTK.

It was confirmed by Nordin et al. (2009) that under a desirable observation environment, VRS-RTK positioning attained a plane precision of 4 cm and height precision of 6 cm. However, this technique occasionally required excessively initial observation time as well as attained unsatisfied precision in the obstructed areas. It was asserted by Hu et al. (2009) that a small number of satellites or a high dilution of precision (DOP) value can impede the initialization of VRS-RTK positioning because with fewer observations and a weak satellite geometric distribution, ambiguity resolution (AR) cannot be correctly determined. Zhang et al. (2006) mentioned that countries such as Germany, the United Kingdom, and Japan have deployed sufficiently dense distributions of reference stations to ensure that the maximum baseline length between a user and a nearby reference station to be less than 40 km. Thus, the dual frequency receivers can then be employed to quickly determine AR. However, as the baseline length increased, the residual of tropospheric modelling in the double difference GPS observations increased, making AR more difficult to solve (Pan et al. 2015). Because the tropospheric delay effects influence global positioning system elevation positioning, and the water vapor in the atmosphere can't be eliminated effectively is also the important reason. (Wang. 2009; Yeh et al., 2013).

Compared with the conventional static GPS observation, which requires relatively more operators and instruments, an e-GPS service with VRS-RTK positioning technique is advantageous for its efficient personnel allocation and reduced operating time. Moreover, the e-GPS system in Taiwan constructed in 2007 has been verified to exhibit a plane precision of 2 cm and height precision of 5 cm (Lin et al., 2007). Nevertheless, several locations covered by the system still cannot be effectively positioned using VRS-RTK because of communication difficulty in the mountain areas.

Similar to VRS-RTK, post-processing mode of kinematic using VRS data (VRS-PPK) is also attractive for operations that do not require any reference station setups. Furthermore, this technique does not require real-time communication device. Through a VRS system, this technique expends less operating time than does the static observation; hence, it is potentially applicable for conducting horizontal densification of control stations used by cadastral surveys. Therefore, this study adopted the GPS static data obtained through the control station resurvey in 2012 (administered by the Ministry of the Interior) to examine the internal precision and external accuracy of VRS-PPK positioning, thereby providing application standards for using this technique to conduct horizontal densified surveying or resurvey of control stations.

1.1 VRS Observations

To overcome the bottleneck of the conventional RTK technique, the VRS-RTK positioning technique has been developed in recent years. Generally, VRSs are established in a continuously operating reference station system. This enables not only conducting correction through an regional error model but also generating VRS observations using location transmitted from any rover in the service area. The position of a VRS lies extremely close to that of a rover, and the distance-correlated errors can be eliminated; hence, the shortest baseline length between the VRS and rover can provide a highly accurate coordinate solution of the rover.

A network connects numerous continuously operating reference stations to compile and transmit satellite observations to a control and computation centre for data inspection and storage, and regional error corrections are calculated accordingly. Through a filter estimation method, the integer ambiguities of all the double difference observations in the reference network can be solved, and equation (1) can be used to correct the overall systematic errors of each reference station relative to the master station. Accordingly, an error modelling is constructed:

$$\mathrm{Corr}^{k,l}_{MS,i} = \Phi^{k,l}_{MS,i} - \rho^{k,l}_{MS,i} - \lambda N^{k,l}_{MS,i} = \left(\frac{I^{k,l}_{MS,i}}{f2}\right) + T^{k,l}_{MS,i} + \Delta\rho^{k,l}_{MS,i} \quad (1)$$

where $\mathrm{Corr}^{k,l}_{MS,i}$ is the correction of the double difference error of reference station i relative to the master station (MS). Superscript k, l denotes satellite number. $\Phi^{k,l}_{MS,i}$ represents carrier phase observation. $\rho^{k,l}_{MS,i}$ pertains to geometrical distance. $\lambda N^{k,l}_{MS,i}$ is equal to wavelength multiplied by integer ambiguity. $\left(\frac{I^{k,l}_{MS,i}}{f2}\right)$ represents ionospheric correction. $T^{k,l}_{MS,i}$ denotes tropospheric correction, and $\Delta\rho^{k,l}_{MS,i}$ pertains to orbital error correction.

After a point positioning coordinate transmitted from a rover is received, the satellite observations and error corrections of a VRS are calculated through interpolation. Within the coverage area of the reference network, the error obtained from each reference station can be adopted to solve the systematic errors of any position in the coverage area through interpolation. Because the systematic errors of a satellite observation are mostly distance-dependent and highly correlated with the relative positions of the reference stations in the network area, a polynomial can be used to describe the relationship between systematic errors and reference station positions. For example, equation (2) employs the bilinear surface method:

$$\mathrm{Corr}_{VRS} = a \cdot \Delta E_{VRS} + b \cdot \Delta N_{VRS} + c \cdot \Delta E_{VMS} \Delta N_{VRS} \quad (2)$$

where Corr_{VRS} is the correction for the double difference error between a reference station and a virtual reference station; a, b, and c are the coefficients of the polynomial; and $\Delta E_{VRS}, \Delta N_{VRS}$ is the plane coordinate difference between any site and the reference station. As the equation involves three interpolation parameters, at least three reference stations must be used to conduct interpolation calculation. If the number of reference stations can be further increased, the least squares method can be adopted.

After the interpolation process is used to determine the correction for the double difference error of VRS relative to the master station, information based on the coordinate of VRS and satellite orbits can be used to determine the double difference virtual observation between the two stations:

$$\Phi^{k,l}_{MS,VRS} = \rho^{k,l}_{MS,VRS} + \mathrm{Corr}_{VRS} + \lambda N^{k,l}_{MS,VRS} = \Phi^{k}_{MS} - \Phi^{l}_{BS} - \Phi^{k}_{VRS} + \Phi^{l}_{VRS} \quad (3)$$

Finally, the coordinate of a rover is solved on the basis of the double difference observation between the rover and a VRS. Equation (4) assumes that k denotes the reference satellite that accounts for the double difference, and that the VRS observation only involves the geometric range and clock errors. Accordingly, the observation sent from VRS to other satellites can be obtained using equation (5):

Observation of reference satellite:

$$\Phi^{k}{}_{VRS} = \rho^{k}{}_{VRS} + c(dt_{VRS} - dt^{k}) \quad (4)$$

Observation of other satellites:

$$\Phi^{l}{}_{VRS} = \rho^{k,l}{}_{MS,VRS} + Corr_{VRS} + \lambda N^{k,l}{}_{MS,VRS} - \Phi^{k}{}_{VRS} \quad (5)$$

In equation (4) and equation (5), systematic error correction is not applied in the observation of the reference satellite; instead, it is added to the observations of other satellites. However, during the process of solving for the position of a rover, the double difference observation of the rover is applied to ensure adequate correction of systematic errors.

Ⅱ. DATA PROCESSING AND ANALYSIS

2.1 Testing Data And Specifications

This study compiled GPS observations from 549 testing sites in the resurvey area covered by control stations in middle and western Taiwan. The resurvey was administered by the Ministry of the Interior in 2012. The total data length was 6 hours (9:30 to 15:30 local time), with a data interval of 5 seconds per observation.

To investigate topics related to VRS-PPK, the data were further divided into three 1-hour sessions, namely the morning (9:30 to 10:30), noon (11:30 to 12:30), and afternoon (14:30 to 15:30) sessions. The VRS observations corresponding to each session were generated through Trimble GPSNet system. Next, a post-processing was used to calculate the VRS-PPK coordinates and evaluate the internal precision and external accuracy of VRS-PPK solutions.

The standard deviations of the coordinate solutions, differences between the coordinates obtained through repeated observation, and differences between the converted baseline lengths were examined to discuss the applicability of VRS-PPK to horizontal densified surveying. The precision and accuracy indicators are described as follows:

(1)Standard deviations of the coordinates in each session

According to the VRS-RTK Densified Control Surveying Handbook of the NLSC (NLSC, 2010), the densified control points should be configured and observed through a data interval of 1 Hz, and the data should be continuously retrieved for at least 20 minutes. Regarding the relevant office work, the observation data (in RINEX format) was post-processed using an on-the-fly (OTF) mode to calculate the coordinate in each epoch. Subsequently, a normal distribution of 99% confidence interval was used to detect the outliers, and the coordinates lying more than 3 σ (standard deviation) away from the mean were excluded. The exclusion rate should not exceed 50%, and the σ of each session must be less than 2 cm in both N and E (horizontal) components and less than 5 cm in vertical component. The coordinates that fulfilled this criterion were averaged as the positioning outcome, or the final coordinate of the testing site.

(2)Differences between the coordinates in the repeated observations

To validate the outcomes of repeated observations, each testing site must be repeatedly observed through two sessions that are at least 1 hour apart from each other. In each session, the observation must contain the data of at least 180 fixed solutions. Next, the difference between the averaged coordinates of the two sessions must be less than 2 cm in both N and E components. Otherwise, additional observations were collected until the difference between two sessions fulfils this criterion. Next, the two sets of observation were averaged as the final coordinate of the testing site.

(3)Differences between the converted baseline lengths

After the coordinates were validated using the aforementioned internal precision method, the external accuracy of the coordinates were also examined. Specifically, a baseline combination was used to derive the horizontal baseline length from the VRS-PPK coordinates of any two selected testing sites. Next, it was compared with that of each testing site coordinate determined through the network adjustment of the 6-hour static data. According to the resurvey standards of Level 3 control points in Taiwan (Chang, 2001), a 95% confidence interval was adopted to ensure that the baseline length (L) difference must be less than 30 mm + 6 ppm × L. Because the baseline lengths were compared using the identical geodetic datum, problems related to datum transformation and scaling were not existed.

2.2 Computation And Analysis

A free translation, editing and quality checking (TEQC) software, developed by UNVACO for managing GPS data (UNAVCO, 2016), was employed to convert and edit the raw GPS data collected in this study. However, because a substantial amount of raw data were collected, the related interface programs for processing the raw data in batches was coded using Visual Basic. An open source software RTKLIB developed for GPS positioning was adopted to post-process the kinematic solutions of each testing site (Takasu, 2016). Finally, Bernese software version 4.2 and the coded visual graphic interface were used to execute the network adjustment of the static data (Hugentobler et al., 2001).

Ⅲ. STANDARD DEVIATIONS OF THE COORDINATES IN EACH SESSION

The testing area consisted of 549 sites. Dividing each site into three sessions yielded a total of 1647 data sets. The coordinate solutions of each site were calculated using the VRS-PPK technique, and then the outliners were removed through the iteration until the remaining kinematic solutions fulfilled the horizontal and vertical thresholds. Table 1 illustrates the number of testing sites that were successfully positioned, and the standard deviations of the coordinate solutions.

Tab. 1 Plane coordinate solution of each session

Session		Morning	Noon	Afternoon
Testing site (pts)		549	549	549
Successfully positioned site (pts)		400	392	366
Success rate		73%	71%	67%
Standard deviation (cm)	Min	0.3	0.4	0.2
	Max	3.9	3.9	5.3
	Average	1.1	1.3	1.4

After the standard deviations of the coordinate solutions were considered, the success rate of positioned test site ranged between 67% and 73%, with an average success rate of 70%. The morning session attained the most favourable success rate, whereas the afternoon session attained the least favourable success rate. The success rates of these two sessions differed by approximately 6%. The standard deviations of plane coordinate exceeding the threshold criterion (2 cm) occurred in each session. The morning session attained the smallest average standard deviation, whereas the afternoon session attained the largest average standard deviation. The average standard deviation of the three sessions was 1.3 cm.

In this study, the testing sites were observed in the summer and early autumn periods, during which the level of atmospheric disturbance was high in the afternoon. Consequently, GPS signals observed in the afternoon were easily interfered, resulting in a higher exclusion rate of undesirable positioning solutions.

Fig. 1 illustrates all the testing sites (including both the success and failure sites). Observing the distributions and surrounding environments of the sites revealed that the failure sites were mostly located in the lower mountains and valleys of middle and eastern Taiwan, and several failure sites were located in the high-elevation regions of the Central Mountain Range.

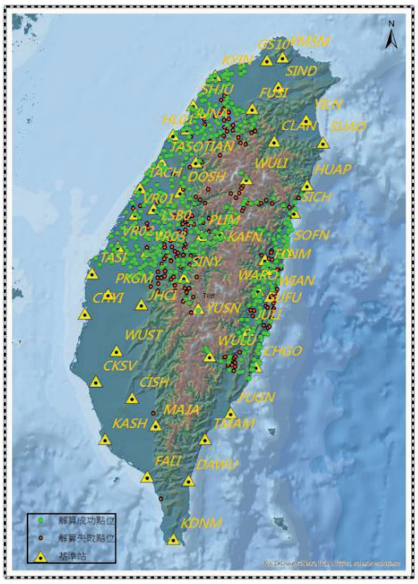

Fig.1 Testing site distribution (yellow triangles denote reference stations; green dots denote successfully positioned sites; red dots denote failure sites)

Receiving complete observation data is difficult in the lower mountain regions because dense trees prevent satellite signal penetration. In valley regions, steep slope decreases the number of visible satellites. In high-elevation regions of the Central Mountain Range, although an unobstructed sky view is available, the amount of data provided for VRSs can be affected by a lower elevation or distribution of reference stations (as seen in Fig.2). Therefore, in this study, the coordinates determined through VRS-PPK in these regions were inadequate.

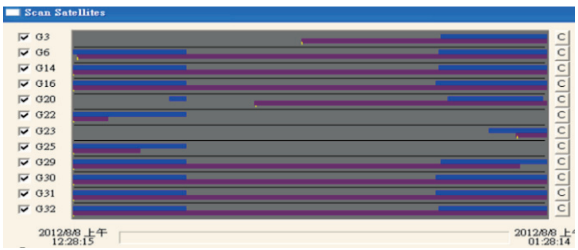

Fig.2 Interruption of VRS observations in high-elevation testing sites (as indicated by the red box)

Examining the standard deviations of coordinates at the testing sites showed that 405 sites and 1158 data sets met the criterion regarding how the observations of a site in two sessions must fulfil the precision criterion. These testing sites were used for subsequent analyses.

Ⅳ. DIFFERENCES BETWEEN THE COORDINATES IN THE REPEATED OBSERVATIONS

According to the VRS-RTK handbook of the NLSC, the coordinate difference between the two data sets at a new control point should be less than 2 cm in horizontal component before the two sets is averaged to determine the final coordinate of the control point. In this study, the differences between the two observation sets of the 405 testing sites were calculated. Fig. 3 illustrates the absolute set differences of the testing sites. Fig. 4 and Fig. 5 display the bar graphs that describe the set differences and the distributions of the testing sites in N component and E component, respectively.

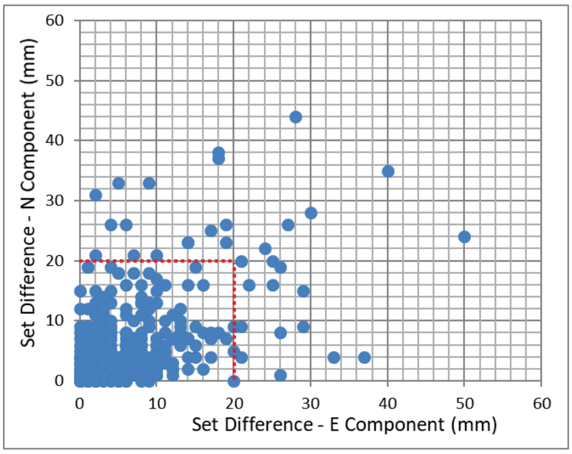

Fig. 3 Distribution of the absolute set differences of the testing sites

As shown in Fig. 3, 34 testing sites attained an absolute set difference exceeding 2 cm in either N or E component. In other words, 92% of the testing sites fulfilled the set difference criterion. When the tolerance for set difference was increased to 3 cm, the number of testing sites not fulfilling the criterion was reduced to 11, meaning that 97% of the testing sites fulfilled this criterion. When tolerance for set difference was further increased to 4 cm, the number of testing sites not fulfilling the criterion was reduced to only 3, or that 99% of the testing sites fulfilled this criterion. Individually observing the set difference in each horizontal component (Fig. 4 and Fig. 5) showed that approximately 5% of the testing sites in both horizontal components did not fulfil the set difference criterion of 2 cm.

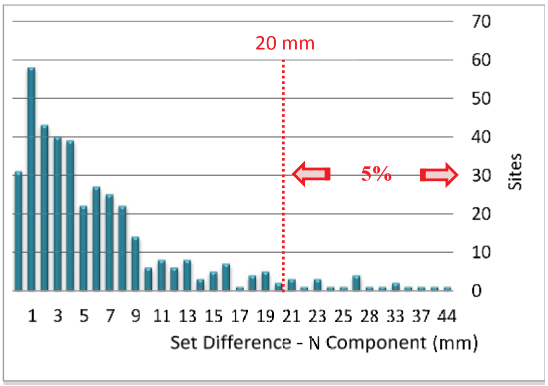

Fig. 4 Distribution of the absolute set differences of the testing sites in N component

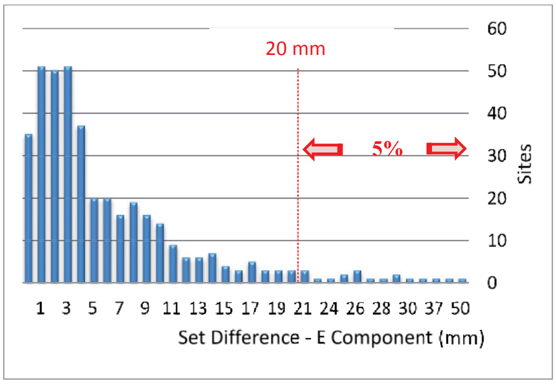

Fig. 5 Distribution of the absolute set differences of the testing sites in E component

Examining the set differences of the testing sites indicates that when VRS-PPK is used to conduct horizontal densified surveying using a 95% confidence interval (2), the tolerance for set difference is suggested to be increased from 2 cm to 3 cm in both horizontal components; in other words, a surveying outcome is considered acceptable when the set difference of each site is less than 3 cm in both the N and E horizontal components.

V. DIFFERENCES BETWEEN THE CONVERTED BASELINE LENGTHS

To evaluate the correctness of the location of each observation site in actual space, the difference between the observed coordinate and known coordinate of a site can be examined. In this study, a baseline-difference approach was adopted to prevent systematic errors caused by differences in geodetic system or processing software. According to the standards of known site inspection described in the VRS-RTK handbook of the NLSC, the threshold criterion was set as that the baseline length (L) difference between two sites must be less than 3 mm + 6 ppm × L. Accordingly, the adequacy of the surveying outcomes was evaluated.

The network adjustment solutions of the static observation data were adopted as the reference coordinates, whereas the 405 VRS-PPK sites fulfilling the precision criteria were employed as the testing coordinates. MATLAB was used to code a Delaunay triangulation program and connect the sites into triangular networks. The neighbouring sites formed 1200 independent baselines (Fig. 6). This enabled comparing the differences between the lengths of the horizontal baselines converted from the VRS-PPK coordinates and static-based coordinates. Fig. 7 illustrates the distribution of the baseline length differences.

The root mean square of the baseline length differences was 1.8 cm. As shown in Fig. 7, 6, 11, and 39 baselines attained an absolute difference greater than 10 cm (0.5%), 8 cm (2%), and 7 cm (3%), respectively, indicating that when comparing the coordinates obtained through VRS-PPK with those obtained through static observation, the difference between the baseline lengths of less than 7 cm was within a 95% confidence interval.

When the baseline length (L) was considered and the threshold was set as 30 mm + 6 ppm × L, 16 baselines (1%) did not fulfil this criterion. The sites associated with these baselines were widely distributed and not concentrated in a specific region. In addition, most of these sites also did not fulfil the set difference criterion. When the threshold was restricted to 20 mm + 4 ppm × L (standard for Level 2 control point resurvey), 33 baselines did not fulfil the criterion, although the passing rate reached 97%. When the threshold was further restricted to 10 mm + 2 ppm × L (standard for Level 1 control point resurvey), 95 baselines did not fulfil the criterion, with the passing rate decreasing to 92%.

Examining the baseline length differences indicates that when VRS-PPK is employed to conduct horizontal control station resurvey using a 95% confidence interval (2), the threshold criterion for baseline length difference can be tightened up to 20 mm + 4 ppm × L.

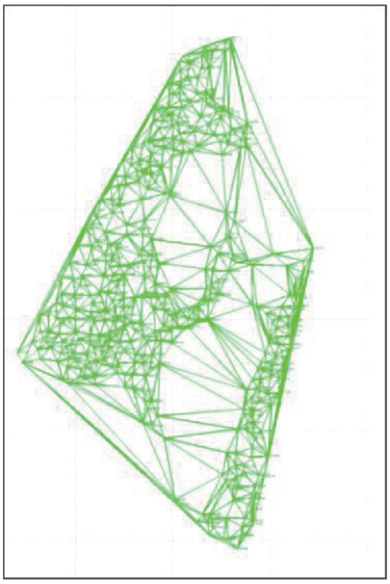

Fig. 6 Baseline network formed by the testing sites

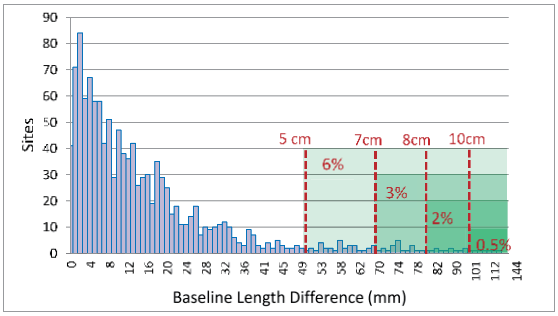

Fig. 7 Distribution of the baseline length

VI. CONCLUSIONS AND SUGGESTIONS

In this study, the static GPS observations obtained through the control station resurvey campaign carried out in 2012 were compared with the observation data generated through VRS to examine the internal precision and external accuracy of VRS-PPK, thereby evaluating whether the current VRS-RTK operating standards employed by the Ministry of Interior of Taiwan is applicable to VRS-PPK solutions. Moreover, the results can serve as a reference for formulating operating standards for administering horizontal densified surveying or control station resurvey using VRS-PPK. According to the test results, the following conclusions and suggestions are proposed:

(1) The standard deviation of 2 cm in horizontal component and 5 cm in height component was used as the criterion to screen for positioning solutions determined through VRS-PPK. The results showed that the average success rate of VRS-PPK was approximately 70%. The morning session attained the highest success rate, whereas the afternoon session attained the lowest success rate. The success rates of the morning and afternoon sessions differed by 6%. The overall plane coordinate precision was 1.3 cm.

(2) Most of the 144 testing sites eliminated through the standard deviation criterion were located in the lower mountains and valleys of middle and western Taiwan as well as in high-elevation regions of the Central Mountain Range. In the lower mountain and valley regions, dense trees and steep slope decrease the number of visible satellites. In high-elevation regions of the Central Mountain Range, although an unobstructed sky view is available, the amount of available data for VRSs might be affected by elevation or a low distribution of reference stations; hence, the VRS-PPK solutions obtained in these regions are inapplicable.

(3) The set difference of each testing site was determined by the difference between the average coordinates of two observation sets. The results revealed that 8% of the testing sites did not fulfil the criterion that the set difference should not exceed 2 cm in the horizontal component; hence, according to a 95% confidence interval, this outcome was undesirable. By contrast, 97% of the testing sites could fulfil the set difference criterion when the tolerance was increased to 3 cm in both horizontal components. Consequently, this value is applicable to evaluating the solutions of horizontal densified surveying conducted through VRS-PPK.

(4) After the standard deviation and set difference of each testing site was used to evaluate the solutions of VRS-PPK, root

mean square errors were employed to examine the baseline length difference between the VRS-PPK coordinates and static network solutions. When the threshold criterion for baseline length difference was restricted to 20 mm + 4 ppm × L, 97% of the baselines could fulfil this criterion; hence, it can be used as the standard for assessing the solutions of control station resurvey conducted through VRS-PPK.

(5) When using standard deviation and set difference to screen for VRS-PPK solutions, the observation period of each set (1200 observations in 20 munities) needs to be extended because the passing rate of 70% achieved through the condition of this study was undesirable. Alternatively, the observation can be increased to more than 3 data sets to lower the failure rate. However, doing so will diminish the temporal advantage of VRS-PPK.

(6) When VRS-PPK was used to provide the coordinates, several testing sites in the mountainous regions could not generate complete VRS data, resulting in unsolvable data. Failure to identify such a problem during a fieldwork operation can render the observation unsuccessful. Therefore, when VRS-PPK is adopted to conduct horizontal densified surveying, VRS-RTK can be also employed adequately during fieldwork operations to realise the quality of positioning, thereby identifying the problem of incomplete VRS data generation before relevant office work is executed.

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