

Precise Real-Time Positioning Using Network RTK

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1. Introduction

In the classic RTK method using a single reference station the rover needs to work within a short range from the reference station due to the spatial decorrelation of distance-dependent errors induced by the ionosphere, troposphere and orbital errors. The operating range of RTK positioning is thus dependent on the existing atmospheric conditions and is usually limited to a distance of up to 10-20 km. In addition, no redundancy of the reference stations is usually available if the reference station experiences any malfunctioning. The constraint of the limited reference-to-rover range in RTK can be removed by using a method known as Network RTK (NRTK), whereby a network of reference stations with ranges usually less than 100 km is used. The network stations continuously collect satellite observations and send them to a central processing facility, at which the station observations are processed in a common network adjustment and observation errors and their corrections are computed. The observation corrections obtained from the network are sent to the user, operating within the coverage area of the network, to mitigate his observation errors.

In this chapter, the principles of Network RTK are first discussed and the advantages and disadvantages of the method are given. Next, the network design parameters are discussed, which include network baseline lengths and configuration, the communication method between the computing centre and the user, and the amount of calculations required by the network processing centre and by the user. Description of possible network processing techniques, their basic models, and a comparison between their advantages and disadvantages are given. Finally, some important NRTK applications are discussed including the use of NRTK in engineering surveying, machine automation and in the airborne mapping and navigation. Results from real-time testing are discussed.

2. Principles of the network RTK

The aim of network RTK is to minimise the influence of the distance dependant errors on the computed position of a rover within the bounds of the network. NRTK provides redundancy of reference stations in the solution, such that if observations from one reference station are not available, a solution is still possible since the observations are gathered and processed in a common network adjustment. Figure 1 illustrates a simple demonstration of the concept of NRTK through representation of the relationship between

the modelled distance-dependent errors and their actual values. The error planes at the three shown reference stations are at different levels. The NRTK provides an error surface formed from the errors at the three reference stations (a plane in this case). The actual change of error between the reference stations is shown in red. If a user is close to any of the stations, assuming having the same level of error of that reference station will give reasonable accuracy and results in small positioning errors at the rover. As the user moves away from the reference station, the magnitude of the differential error between the actual and the reference station error level increases. On the other hand, the differential error between the actual error and the NRTK estimated error, interpolated on the NRTK error surface at the location of the rover when used, is significantly minimised.

In principle, the RTK network approach consists of four basic segments: data collection at the reference stations; manipulation of the data and generation of corrections at the network processing centre; broadcasting the corrections, and finally positioning at the rover utilizing information from the NRTK. In the first segment, multiple reference stations simultaneously collect GNSS satellite observations and send them to the control centre, where a main computer directly controls all the reference stations, mostly via the Internet. All reference stations should use geodetic-grade multi-frequency GNSS receivers. The incoming GNSS observation data from all operating reference stations are screened for blunders and next their ambiguities are fixed. The control computer uses these data in processing a networking solution, and the data are archived for post-processing use. The network information are then broadcast to users. The network information depends on the processing algorithm and may include any of the following: observations from one reference station (physical or virtual), coefficients for interpolation of corrections within the coverage area, and observation corrections at a group of reference stations. To increase reliability, it is recommended to let a second computer work in real time as a backup to the main computer in the event of any malfunctioning.

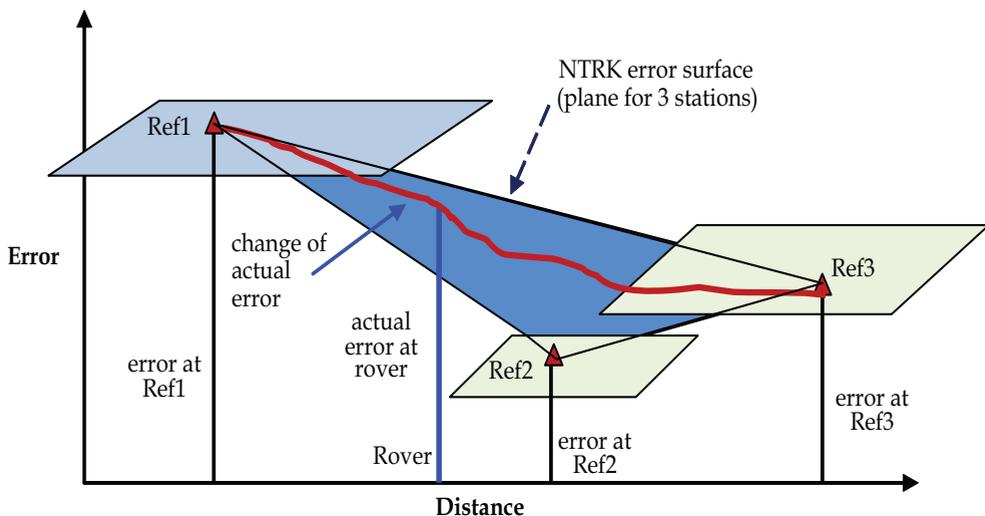


Fig. 1. Relationship between errors in a small NRTK coverage area

NRTK usually requires a minimum of three reference stations to generate corrections for the network area. In general there is no restriction concerning the network size, it can be regional, national, or even international. However, reference station separation is usually restricted to less than 100 km to allow for quick and reliable ambiguity resolution. As the number of stations increases, redundancy increases, and better corrections can be estimated. If one or two reference stations fail at the same time, their contribution can be eliminated from the solution and the remaining reference stations can still provide the user with corrections and give reliable results (El-Mowafy *et al.*, 2003, Hu *et al.*, 2003). Typically, a NRTK server system would consist of the following components (e.g. Leica Geo systems, 2011):

- A site server connected to each reference station receiver,
- A network server that acquires the data from the site servers and sends it to the processing centre,
- A cluster server that hosts the network processing software. The software performs several tasks including: quality check of data, apply antenna phase centre corrections, ambiguity fixing, modelling and estimation of systematic errors, interpolation of errors (corrections) in some techniques (e.g. VRS, PRS) and generation of virtual observations, model coefficients in others (FKP), or Mac data.
- A firewall is usually established to protect the above servers from being accessed by a user.
- RTK proxy server to deal with requests from the users and send back network information.
- The user interface to send/receive data from the NRTK centre.

The main advantages of the Network RTK can be summarised as follows:

- Cost and labour reduction, as there is no need to set up a base reference station for each user.
- Accuracy of the computed rover positions are more homogeneous and consistent as error mitigation refers to one processing software, which uses the same functional and stochastic modelling and assumptions, and use the same datum.
- Accuracy is maintained over larger distances between the reference stations and the rover.
- The same area can be covered with fewer reference stations compared to the number of permanent reference stations required using single reference RTK. The separation distances between network stations are tens of kilometres, usually kept less than 100 km.
- NRTK provides higher reliability and availability of RTK corrections with improved redundancy, such that if one station suffers from malfunctioning a solution can still be obtained from the rest of the reference stations.
- Network RTK is capable of supporting multiple users and applications.

Network RTK has though some disadvantages, which are:

- The cost of subscription with a NRTK provider.
- The cost of wireless communication with the network (typically via a wireless mobile using for instance GPRS technology).
- The dependence on an external source to provide essential information.

3. Network design parameters

Establishing a network RTK usually starts after a thorough cost/benefit analysis. At the design stage, the following main factors should be considered:

1. Baseline lengths (distances between the reference stations) station locations and network configuration (number and geometric distribution of reference stations).
2. The communication method between the computing centre and the user.
3. Calculations required by the network control and by the user (network algorithm).

These factors are discussed in the following sections.

3.1 Distance between the reference stations and network configuration

The main advantage of the network approach is that it improves modelling of the distance-dependent errors over long distances (El-Mowafy *et al.*, 2003 and Euler *et al.*, 2004). The observation corrections (computed as the same value of the errors but with opposite sign) can be generated after removing cycle slips and determining double differenced phase ambiguities between the reference stations. A major technical challenge in NRTK is ambiguity resolution within a reasonable short period over such large distances between reference stations. In order to achieve a fast and reliable ambiguity resolution, the distance between the reference stations is better chosen not to exceed 100 km (Wübbena and Willgalis, 2001). Typically, baseline lengths in NRTK range between 20 km and 100 km (70 km on average).

In principle, a minimum of three stations is required to generate RTK network corrections, but in practice this number should not be less than five. The increased redundancy of reference stations improves positioning accuracy and ambiguity resolution and helps to sustain network availability and reliability in the case of a temporary failure of any reference station. However, the degree of redundancy should be evaluated by means of a cost/benefit analysis, balancing the need to improve the economical aspects of establishing and running the network and keeping the required degree of redundancy. Hence, selection of baseline lengths by the network designers should satisfy the following conditions:

- covering of the whole area of interest with reliable corrections;
- maintaining sufficient station redundancy;
- achieving a reliable ambiguity resolution with acceptable confidence level at every location within the network area as long as a minimum of five satellites are observed;
- Ensuring reliable communications between the reference stations and the network centre (mostly via the internet through land lines, and in remote areas through satellite communication);
- choosing sites free from multipath and radio frequency interference. It is also preferable to have the references stations situated at a similar altitude.

In network configuration, the following can be taken into consideration:

- For a limited number of reference stations, it is recommended to shape the network as a polygon with one or more central stations.
- A compact shape of the network is preferable (i.e. a circular network is better than a rectangular network).

- Some geographic regions (i.e. equatorial or high latitude) will require a denser network than in the mid latitudes due to poorer-satellite geometry, satellite availability, and ionosphere disturbance etc.

In practice, a main factor affecting the choice of station distances and network configuration is finding suitable sites for the reference stations. The main considerations are availability of communication infrastructure and obtaining approval of site owners (whether a government or a private sector).

It is also possible to integrate observation corrections estimated from networks of different sizes. For instance, errors of regional or even global nature, such as satellite orbital errors, clocks and the regional behavior of the ionosphere, which slowly change, can be estimated from regional networks. The local ionospheric and tropospheric errors on the other hand can be estimated from local networks. Thus, RTK networks can be configured such that areas of heavy usage can be covered by a close-meshed reference station network for highest accuracy and reliability in positioning, whereas less important areas are covered by a wide-meshed network of regional or national extension (Wübbena *et al.*, 2001).

3.2 Communication method between the processing centre and the user

Real-time applications require a communication link between a service provider and the user. Currently, there are two main modes of communication that can be used in network RTK; either a duplex (bi-directional) communication or one-direction communication. Each method has its advantages and disadvantages. In choosing which communication method to use, the designer has to consider economical factors such as: operational cost by the user, cost of maintenance of existing infrastructure and/or building new one, and the amount of computations needed by the rover and the processing centre. The technical aspects that need to be addressed include:

- expected signal strength at different locations,
- number of users,
- range and coverage (Wu, 2009) ,
- transmission bandwidth,
- protocol,
- reliability and error correction,
- latency (one second and shorter data transmission latencies are required for cm level positioning accuracy).

At present, the duplex communication mode is the mostly used method. In this mode, a cellular modem such as a General Packet Radio Service (GPRS) or Global System for Mobile Communications (GSM) are used. GPRS is usually preferred as it is more economical than GSM since the user only pays for the data packets received, not for the entire call duration when using GSM. GPRS can provide a stable and reliable connection with latencies less than one second (Hu *et al.*, 2002). The duplex approach has a restriction on the number of users, as this number is limited by the ability of the NRTK processing centre to simultaneously perform calculations for all users. This may also result in extended latency in receiving the network information. For a limited number of users this latency is usually less than three seconds. On the other hand, the one-direction communication method mainly employs VHF or UHF broadcasting or encodes the RTK corrections into a broadcast TV audio sub-carrier

signal (Petrovski *et al.*, 2001). For VHF broadcasting, allocation of suitable broadcast radio frequency and obtaining its license is an important issue in the early development of a network RTK. The main advantage of this method is that there is no restriction on the number of users concurrently using the NRTK service. However, the main disadvantage of the method is the high cost of the infrastructure needed to build radio signal repeaters, if needed, to cover the whole area. In addition, some problems can be experienced due to the possibility of receiving signals of varying strength in different locations, and possible frequency jamming. A mix of both communication methods is however possible (Crudace *et al.*, (2002).

The data transmission from the reference stations to the control centre server and from the control centre server to the user for RTK corrections is mostly carried out via the Network Transport of RTCM via Internet Protocol (Ntrip), BKG, 2011. Ntrip is an open source and can be downloaded from the internet (LENZ, 2004). Ntrip was built over the TCP/IP foundation and is an application - level protocol for streaming GNSS data over the internet. It was first developed by the German Federal Agency for Cartography and Geodesy (BKG). Ntrip uses HTTP and has three components: Ntrip Client, Server and Ntrip Caster. Ntrip is designed for disseminating differential correction data (e.g. in the RTCM-SC104 format) or other kind of GNSS streaming data to stationary or mobile users over the internet. It allows simultaneous PC, Laptop, PDA, or receiver connections to a broadcasting host. Ntrip supports wireless internet access through mobile IP networks like GSM, GPRS, EDGE, or UMTS (BKG, 2011).

To reduce latency, the amount of data transmitted to the rover should be minimised. One possible solution is to change (optimise) the update rates for the different parameters to follow their physical behaviour. Distance dependent errors can thus be separated into a dispersive component, consisting mainly of the ionospheric refraction, and a non-dispersive component consisting of the tropospheric refraction and orbit errors. Different proposals for optimising the update rates have been made. An update rate of 15 seconds seems reasonable for non-dispersive correction differences, while an update rate of only 10 seconds may be sufficient for the dispersive contribution (Euler *et al.*, 2004). However, the impact of these rates on the Time-To-First-Fix (TTF) of carrier phase ambiguities should be carefully studied, as it lies at the top of the user interests (El-Mowafy, 2005).

The type of communications used also affects the network algorithm and the amount of calculations required at the processing centre and by the user. For instance, if a bi-directional communication is used, the processing centre can individualise the network information for a user based on his/her approximate location. Thus, the computations made at the user receiver are minimised. On the other hand, if the data link is one-directional, the user has to make the necessary interpolation of errors at his location and has to identify a suitable reference station to use.

3.3 NRTK solution methods

Currently several solution methods can be applied in Network RTK, including the Virtual Reference Station (VRS), Pseudo-Reference Station (PRS), individualised Master-Auxiliary corrections (iMAX), Area-Parameter Corrections (Flächenkorrekturparameter -FKP- in its German origin), and the Master-Auxiliary (MAC) method. In VRS, PRS and iMAX

referencing is made to a non-physical reference station located in the vicinity of the approximate position of the rover and virtual observations are generated to refer to this non-physical reference station. The user typically has no information about the size of errors and their behaviour. In contrast to the non-physical network approach, FKP and MAC broadcast raw reference station observations and network information separately. The network information is represented by dispersive and non-dispersive corrections and the rover software decides how the network information is applied. A summary of these methods is given in section 5.

Once the network errors are computed at the reference station, distance-dependent errors need to be interpolated at the location of the user receiver. Several methods can be used for such interpolation process including: the use of linear interpolation, using a linear combination model, applying an inverse-distance linear interpolation or a low-order surface model (used for example in the FKP technique), utilisation of the least-squares collocation approach, or using Kriging techniques (see for instance Fototopoulos, 2000, Dai *et al.*, 2001, Wu, 2009 and Al-Shaery *et al.*, 2010).

4. Estimation of the dispersive and non-dispersive errors at the network reference stations

The mathematical equation of the code and phase observations for the receiver (j) and the satellite (s) at time (t) can be written as:

$$C_j^s = \left| \bar{R}_j^s \right| + \frac{\bar{R}_j^s}{\left| \bar{R}_j^s \right|} \delta \bar{r}^s + c(\delta t_j - \delta t^s) + T_j^s(t) + \delta T_j^s(t) - I_j^s(t) - \delta I_j^s(t) + p_j^s + \varepsilon_{p_j}^s \quad (1)$$

$$\phi_j^s = \left| \bar{R}_j^s \right| + \frac{\bar{R}_j^s}{\left| \bar{R}_j^s \right|} \delta \bar{r}^s + c(\delta t_j - \delta t^s) + T_j^s(t) + \delta T_j^s(t) - I_j^s(t) - \delta I_j^s(t) + \frac{c}{f} N_j^s + p_j^s + \varepsilon_{\phi_j}^s \quad (2)$$

Where:

- C_j^s, ϕ_j^s code and phase observations, respectively;
- $\left| \bar{R}_j^s \right|$ geometric range between the user's antenna and the satellite;
- $\delta \bar{r}^s$ orbit error;
- c speed of light;
- $\delta t_j, \delta t^s$ receiver and satellite clock errors, respectively;
- $T_j^s(t)$ modelled tropospheric refraction delay (mainly the hydrostatic, dry component of the troposphere);
- $\delta T_j^s(t)$ residual tropospheric refraction delay (mainly the unmodelled wet troposphere);
- $I_j^s(t)$ modelled ionospheric refraction delay if applied (frequency dependent);
- $\delta I_j^s(t)$ residual ionospheric refraction (frequency dependent);
- f signal frequency;
- N_j^s integer phase ambiguity;
- p_j^s total site dependent errors (antenna phase centre and multipath δM_j^s);
- $\varepsilon_{p_j}^s, \varepsilon_{\phi_j}^s$ code and phase remaining random noise, respectively.

From the above equations, one can see that positioning accuracy from GNSS phase observations is limited by two types of errors: the distance dependent errors, which include orbit, ionosphere and troposphere errors, and station dependent errors, which include multipath, antenna phase centre variation, and receiver hardware biases. The network estimation methodology uses the known information of the antennae and site to reduce station related errors and focuses on estimating the distance-dependent errors.

For the station-dependent errors, multipath can be minimised using choke rings and modelling the site specific multipath pattern taking advantage of the fixed reflector to antenna geometry at reference stations and of the daily repeatability of multipath. This can be done utilizing techniques such as the Hilbert Huang transformation to decompose the time-shifted post-fit GPS phase signal residuals (Hsieh and Wu, 2008). Another approach is to include multipath error in the network estimation process, which will average out the uncorrelated multipath errors. To minimise the antenna phase centre variation, the definition of the network reference stations antennae has always to be consistent. This can be done by using the same antenna model type for all reference stations and unifying antenna orientation. To eliminate the phase centre variation, an absolute calibration of each antenna is recommended. However, most current networks only apply relative calibration of the antennae, which is a standard calibration process that can be applied for the type of antenna used, determined relative to a reference antenna (typically a Dome Margolin Model T with choke ring).

The distance dependent errors can be separated into a dispersive component (i.e. frequency dependent), which is the error induced by the ionosphere, and a non-dispersive component, that include orbital and tropospheric errors. Estimation of the dispersive and non-dispersive errors at the network reference stations can be performed in several ways. In one approach the state of individual GPS errors in real time can be estimated by processing all stations of the network simultaneously using un-differenced observables (Wübbena and Willgalis, 2001, Zebhauser et al., 2002, Wübbena et al., 2005). Then, the state vector (\bar{X}) at station j reads:

$$\bar{X} = \left(N_j^s, \delta t_j, \delta t^s, \delta \bar{r}^s, \delta I_j^s, \delta r_{j1}^s, \delta M_j^s \right)^T \quad (3)$$

The orbital and tropospheric errors are combined to form the geometric (non-dispersive) error δr_{j1}^s , the ionospheric dispersive error δI_j^s (replacing the terms I_j^s and δI_j^s and in Equations 1 and 2). The state space approach has some advantages; the main one is its ability to constrain each bias by specific models (Wübbena and Willgalis, 2001). Also, a change in the network configuration caused by the breakdown of one of the reference stations can be compensated without much effort. Moreover, in the case of irregular conditions of one of the state parameters, warnings can be issued to the users.

Another popular method for estimation of network errors is using single difference linear combination of observations. The dispersive and non-dispersive components are determined for satellite s and between the reference stations j and k , using dual-frequency receivers of L1 and L2, as follows:

$$\delta \Delta r_{jk,1}^s = \left(\frac{f_2^2}{f_2^2 - f_1^2} \delta \Delta r_{jk}^s \right)_{L1} - \left(\frac{f_2^2}{f_2^2 - f_1^2} \delta \Delta r_{jk}^s \right)_{L2} \quad (4)$$

$$\delta\Delta r_{jk_{o,1}}^s = \left(\frac{f_1^2}{f_1^2 - f_2^2} \delta\Delta r_{jk}^s \right)_{L1} - \left(\frac{f_2^2}{f_1^2 - f_2^2} \delta\Delta r_{jk}^s \right)_{L2} \quad (5)$$

5. Summary of network RTK processing techniques

In this section, the most common network RTK techniques used at present are discussed, namely: the VRS, FKP, and Mac.

5.1 The Virtual Reference Station (VRS) method

The VRS technique is currently the most popular NRTK method due to the fact that it does not require changes in the user software, i.e. it is compatible with existing software. The rover applies the standard differential positioning of its observations with observations from a 'virtual' reference station. The distance-dependent errors are computed for each pair of satellites, and for each master-to-another-reference station. The VRS method requires bi-directional communication. The rover sends its approximate position via a wireless communication link (typically a cellular modem in NMEA format) to the network processing centre where computations are carried out for each user (Vollath *et al.*, 2000, Hu *et al.*, 2003). Some network providers use only the nearest three-to-five reference stations to compute the measurement errors for a specific user, see Figure 2. The estimated network measurement distance-dependent errors are interpolated for a virtual reference station (VRS). The VRS location is typically selected at the initial approximate position of the rover. For a kinematic user, this VRS location is kept to preserve the ambiguity values determined from its solution until the range between the VRS and the actual position of the rover becomes too long for precise differential positioning. Then, a new VRS at the most recent position of the user is established.

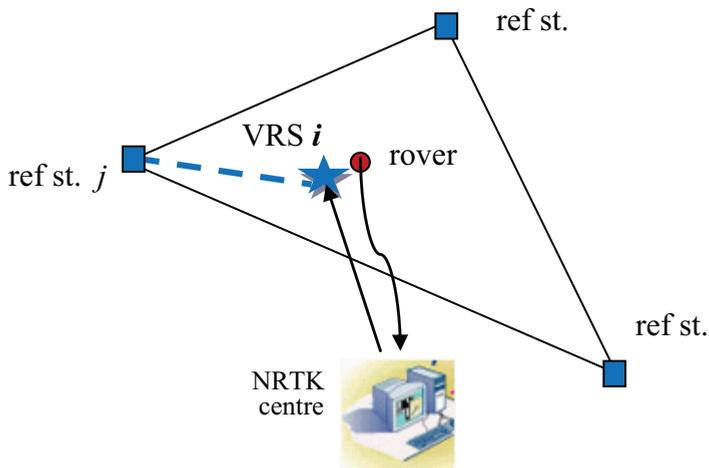


Fig. 2. VRS concept

To construct the observations at the VRS, the VRS and the satellite known positions are firstly used to compute the range between the satellite and the VRS. Similarly, the range between the satellite and the master station is computed, where the master station is usually

selected as the nearest continuously operating reference station to the user. The range difference between the VRS (point i) and master station (point j) with respect to satellite (s) reads:

$$\Delta R_{ij}^s = R_j^s - R_i^s \quad (6)$$

Where:

R_i^s satellite to VRS range
 R_j^s satellite to the master station range
 ΔR_{ij}^s range difference

The interpolated distance-dependent errors, dispersive $\delta\Delta r_{ij_1}^s$ and non-dispersive $\delta\Delta r_{ij_0}^s$ or their total, at the VRS are added to the master station's observations to generate the VRS observations on a satellite-by-satellite and epoch-by-epoch basis for L1 and L2 frequencies, such that:

$$\phi_{ij}^s = \phi_j^s + (\Delta R_{ij}^s - \delta\Delta r_{ij_1}^s + \delta\Delta r_{ij_0}^s + \Delta T_{ij}^s)/\lambda \quad (7)$$

$$P_{ij}^s = P_j^s + \Delta R_{ij}^s + \delta\Delta r_{ij_1}^s + \delta\Delta r_{ij_0}^s + \Delta T_{ij}^s \quad (8)$$

where ΔT_{ij} represents the difference in the modelled part of the troposphere, which is usually subtracted before computation of the network errors, and thus need to be reconsidered. Previous testing of the VRS systems for kinematic ground surveying showed that system positioning accuracy was typically 1-2 cm in plane coordinates and 3-5 cm in height (El-Mowafy *et al.*, 2003). The raw measurements or their corrections are sent to the rover in RTCM version 2.1 format using message types 18/19 or 20/21, respectively. If the former types are used, the constructed VRS observations are computed by the network centre whereas if the latter message types are used, the user has to compute the VRS observations.

A variation to the VRS concept is the Pseudo Reference Station approach (PRS), where the virtual reference station is taken at a pre-selected grid point instead of the approximate position of the user. The virtual observations in this case will also refer to a non-physical reference station. At start of the survey, the baseline length is typically a few metres for the VRS approach and could be several kilometres for the PRS method.

The basic advantages of the VRS mode are that it does not need software changes in the user receiver, and no special formats and conventions are needed. However, a main drawback of the method is the presence of a restriction on the number of users according to the capacity of the network processing centre due to the fact that VRS observations are customised for each user. For kinematic applications, re-determination of VRS may be needed according to the distance between the user and the VRS. In addition, if RTCM message types 18/19 are used, the user will have no information about error sizes, which always helps in interpretation and analysis of positioning results.

5.2 The Area-Parameter Corrections-Flächenkorrekturparameter (FKP) method

The Flächenkorrekturparameter (FKP) or "Area-Parameter Corrections" method represents the network information using coefficients of a surface centred at the location of a physical reference station (Wübbena and Bagge, 2006). Raw reference station observations and

network information are broadcast separately. The method requires advanced software in the rover receiver to do interpolation of corrections. However, unlike the VRS approach, the user has information about error sizes, which helps in quality control and analysis of results. The rover software decides how the network information is applied. For instance, the user can apply the corrections at his location to mitigate observation errors and do differential positioning with the broadcast master station data. Alternatively, the user can utilise the network information to construct a VRS at a nearby location, or the user may apply the Precise Point Positioning (PPP) in what is known as PPP-RTK (Teunissen et al., 2010). FKP can apply an open message with one-directional communication (from centre to users) to cover a certain area. In this case, no restrictions would exist on the number of users. A bi-directional communication can also be employed.

In FKP, the residuals at the network reference stations are assumed to define a surface which is “parallel” to the WGS-84 ellipsoid in the height of the reference station. For baselines less than 100 km, the spatial variations of the residuals can be approximated by a low-order surface model, e.g. a plane, using a bilinear polynomial in the form:

$$\delta r(t) = a(t) (\phi - \phi_R) + b(t) (\lambda - \lambda_R) + c(t) \tag{9}$$

where:

a, b, c coefficients defining the plane at time (t). a and b model the trend of change of error within the area, and c is used for modelling the station specific errors of the master station if undifferenced observations are used or the averaged value of the station specific errors of all the stations if double difference observations are used (Wu et al., 2009).

ϕ, λ geographic coordinates of the interpolated point (in radians)

ϕ_R, λ_R geographic coordinates of the reference point (in radians)

The coefficients are estimated from a weighted least squares solution from the computed residuals at each reference station using Equations 3 or 4 and 5. For instance, for *n* number of reference stations we have:

$$\begin{bmatrix} \delta r_{R-1} \\ \delta r_{R-2} \\ \vdots \\ \delta r_{R-n} \end{bmatrix} = \begin{bmatrix} \Delta \lambda_{R-1} & \Delta \phi_{R-1} & 1 \\ \Delta \lambda_{R-2} & \Delta \phi_{R-2} & 1 \\ \vdots & \vdots & \vdots \\ \Delta \lambda_{R-n} & \Delta \phi_{R-n} & 1 \end{bmatrix} \begin{bmatrix} a \\ b \\ c \end{bmatrix} \tag{10}$$

where $\Delta \lambda_{R-j}$ and $\Delta \phi_{R-j}$ are the difference in latitude and longitude between the reference station R and station j, respectively.

The least-squares estimates for the coefficients can be obtained by (Wu, 2009):

$$\begin{bmatrix} \hat{a} \\ \hat{b} \\ \hat{c} \end{bmatrix} = (A^T A)^{-1} A^T \widetilde{\delta r} \tag{11}$$

Where:

$$A = \begin{bmatrix} \Delta \lambda_{R-1} & \Delta \phi_{R-1} & 1 \\ \Delta \lambda_{R-2} & \Delta \phi_{R-2} & 1 \\ \vdots & \vdots & \vdots \\ \Delta \lambda_{R-n} & \Delta \phi_{R-n} & 1 \end{bmatrix} \text{ and } \widetilde{\delta r} = \begin{bmatrix} \delta r_{R-1} \\ \delta r_{R-2} \\ \vdots \\ \delta r_{R-n} \end{bmatrix} \tag{12}$$

Typically in FKP, two planes are computed for each satellite, centred at each reference station, one plane for the dispersive and another for the non-dispersive corrections. The corrections at the rover are determined through interpolation using the inclination parameters of the correction planes. Results of Euler *et al.*, 2002, showed that the plane surface model gave good results when modelling the regional trends of the correction differences. Although low-order (linear-plane) surface models are usually utilised, longer baselines between reference stations may require polynomials of a higher order.

The surface area model was discussed in several publications, e.g. Varner, 2000, Fotopoulos and Cannon, 2001 and Wübbena and Bagge, 2002. An example is given by the latter study for generation of FKP, where the errors are computed as follows:

$$\delta r_o(t) = 6.37 (FKP_{N_o} (\phi - \phi_R) + FKP_{E_o} (\lambda - \lambda_R) \cos(\phi_R)) (t) \quad (13)$$

$$\delta r_I(t) = 6.37 \alpha (FKP_{N_I} (\phi - \phi_R) + FKP_{E_I} (\lambda - \lambda_R) \cos(\phi_R)) (t) \quad (14)$$

$$\alpha = 1 + 16 (0.53 - \theta/\pi)^3 \quad (15)$$

Where:

- δr_o estimated non-dispersive geometric (orbital and tropospheric) error
- δr_I estimated dispersive ionospheric error
- FKP_{N_I} The FKP parameter in north-south direction for the ionospheric signal “narrow lane” in ppm
- FKP_{E_I} The FKP parameter in east-west direction for the ionospheric signal “narrow lane” in ppm
- FKP_{N_o} The FKP parameter in north-south direction for the geometric signal “ionosphere-free” in ppm
- FKP_{E_o} The FKP parameter in east-west direction for the geometric signal “ionosphere-free” in ppm
- θ the satellite elevation angle in radians

After interpolating the dispersive and non-dispersive errors, they are combined to generate the range residuals for L1 and L2 frequency observations, which read:

$$\delta r_{i_1}^s = \delta r_o + \frac{f_2}{f_1} \delta r_I \quad (16)$$

$$\delta r_{i_2}^s = \delta r_o + \frac{f_1}{f_2} \delta r_I \quad (17)$$

Where:

- $\delta r_{i_1}^s, \delta r_{i_2}^s$ total measurement errors for the frequencies f_1 and f_2 .
- f_2, f_1 frequencies of L1 and L2 signals.

Finally, the range R, derived from the carrier-phase measurements are corrected as follows:

$$R_{i_1 \text{corrected}}^s = R_{i_1}^s - \delta r_{i_1}^s \quad (18)$$

$$R_{i_2 \text{corrected}}^s = R_{i_2}^s - \delta r_{i_2}^s \quad (19)$$

The drawbacks of the FKP method include the need of the rover to perform interpolation of measurement corrections, possible inconsistency at the edge of two adjacent planes due to the use of the linear plane surfaces, and large data formats are needed. In Radio Technical Commission for Maritime services (RTCM) format version 3.1, FKP corrections can be sent via message types 1034 and 1035 for GPS and GLONASS observations, respectively.

5.3 The Master-Auxiliary (MAC) method

In the Mac approach, the rover sends its approximate position via NMEA format to the network processing centre. The centre determines for this specific user the appropriate master station, which is usually selected the closest reference station to its position, and identifies the auxiliary reference stations. These stations are chosen within a catch circle of a predefined radius (e.g. 70 km) around the rover, and with a pre-set number (e.g. from 3 to 14). Figure 3 illustrates the Mac concept. In one Mac approach, a network RTK of large number of reference stations can be subdivided into clusters (Leica Geo systems, 2011). The processing centre defines the appropriate cluster to a user and defines the appropriate network corrections applicable to that user.

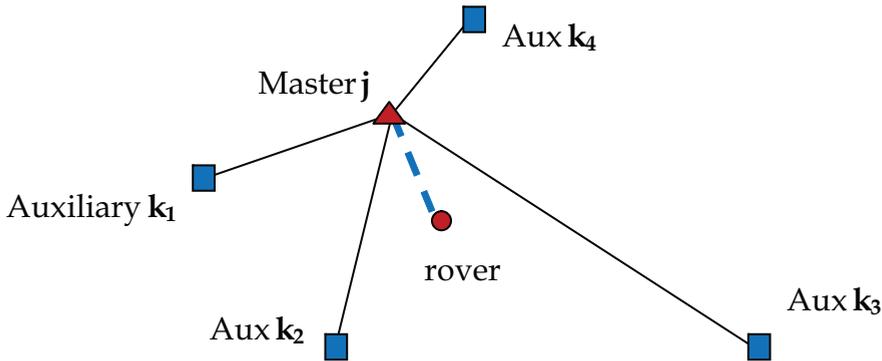


Fig. 3. The Master-Auxiliary NRTK

The rover can receive different types of information according to the strategy used by the Mac processing centre which may include:

- The coordinates and raw measurements of the Master station.
- Measurement corrections at the Master station.
- Correction differences between the Master and Auxiliary stations. These differences when being added to the corrections of the Master will give the corrections at the Auxiliary stations.

The latter Mac corrections can be received via RTCM v3.1 message types 1014-1018, 1030-1031 and 1034-1035 etc.

In Equations (7 and 8), the single observation differences between the Master station j and an Auxiliary station k for satellite s reads (Takac and Lienhart, 2008):

$$\phi_{jk}^s = \phi_j^s + (\Delta R_{jk}^s - \delta\Delta r_{jk_1}^s + \delta\Delta r_{jk_o}^s + \Delta T_{jk}^s)/\lambda \tag{20}$$

$$P_{jk}^s = P_j^s + \Delta R_{jk}^s + \delta \Delta r_{jk_1}^s + \delta \Delta r_{jk_0}^s + \Delta T_{jk}^s \quad (21)$$

One characteristic of the Mac approach is that its data are sent to the user at the same ambiguity level. This can be explained as follows. In the Mac method the carrier phase ambiguities are determined with respect to fixed single difference ambiguity values. However, ambiguity fixing is more reliably performed using a double difference approach. Thus, the ambiguities from the satellite s can be determined from that of the reference satellite q and their double difference as follows:

$$N_{kj}^s = N_{kj}^q + N_{kj}^{q,s} \quad (22)$$

Therefore, the ambiguity bias, which is the difference between the true ambiguity and the estimated ambiguity for the reference satellite, usually known as the ambiguity level, should be estimated. It is common to all estimated ambiguities of satellites observed from one baseline and cancels out in double differencing.

After receiving the Mac information, the rover software is free to decide the method of interpolating the corrections at its location. The processing centre can do the interpolation if needed (individualised I-Max). The rover software is also free on how the Mac information be used to determine its position. For instance, the rover can apply double differencing with the Master reference station as the base. It can also do that after removing the errors from both the Master reference station and its position.

6. PPP- RTK

A more recent direction of NRTK implementation is its integration with the precise point positioning (PPP) technique, Wübbena *et al.*, 2005. In a standalone PPP mode, undifferenced observations are used and the satellite related errors are mitigated by using satellite clock corrections and utilising precise orbits to avoid the orbital errors associated with the use of broadcast ephemeris. These satellite products are typically provided from a processing centre analysing global data such as the International GNSS Service (IGS). Since only one receiver is used in PPP, the ambiguities are solved as part of the unknowns with real numbers and not fixed. As a result, several minutes of data are needed when processing to achieve a reliable convergence of the solution. As the ambiguities are solved as real numbers, only accuracy at the sub-decimetre, at best, is achievable from PPP. However, it is possible to integrate PPP and NRTK into a seamless positioning service, which can provide an accuracy of a few centimetres (Li *et al.*, 2011). The concept of PPP-RTK is to augment PPP estimation with precise un-differenced atmospheric corrections and satellite clock corrections from a reference network, so that instantaneous ambiguity fixing is achievable for users within the network coverage.

A few techniques have been yet proposed for PPP-RTK. In the method presented in Teunissen *et al.*, 2010, un-differenced observation equations for the network stations are used, and thus the design matrix of the network will show a rank defect. This rank defect is eliminated through an appropriate reparametrization (i.e. reduction and redefinition of the unknown parameters). This results in redefined satellite clocks and ambiguities. The tropospheric delay is lumped with the phase and pseudo range satellite clock errors and the ambiguity becomes a between receiver single-differenced ambiguity. Eventually, a full-rank system of observations can be obtained.

In PPP-RTK, the function of the network is to provide the user with satellite clocks and interpolated ionospheric delays. When these precise estimates are passed on to the user, the above given definition of these clocks ensures that the ambiguities of the user are also integer and ambiguity resolution is available at the user side. Satellite clocks for each epoch are added as pseudo-observations, with appropriate variance matrix. The precise IGS orbits are used. For the network processing in Teunissen et al., 2010, a Kalman filter is used, assuming the ambiguities are time-invariant, while for the user, an epoch-by-epoch least-squares processing is used, thus providing truly instantaneous single-epoch solutions. The integer ambiguity resolution of both network and user is based on the LAMBDA method (Teunissen, 1995), with the Fixed Failure Ratio Test (Teunissen and Verhagen, 2009).

7. Network RTK applications

In this section important applications that can benefit from the few cm-level positioning precision and accuracy achievable by using a single GNSS receiver with NRTK are presented.

7.1 NRTK in engineering surveying

Surveying works in construction sites are usually dependent on determination of accurate coordinates and heights. The 3D positioning versatility and accuracy achievable from NRTK encourages the use of this technique for construction surveying works, particularly for large sites when a rapid survey is needed. The method helps in reducing field expenses and time due to reduction of the size of surveying crew, elimination of the need for frequent setups of the surveying instruments, and the reduction of the need for accurate local traverses or multiple control stations within the site. Studies showed that RTK GPS and the traditional techniques employing total stations gave statistically compatible results (El-Mowafy, 2000).

With a typical accuracy of 1-5 cm, the NRTK GPS technique can be utilised for medium accuracy construction survey works such as:

- grading,
- staking out of marks with medium accuracy, such as roads, footings, pipelines, utilities, landscaping, fences etc.,
- cadastral surveys,
- mapping,
- checking of the as-built structures,
- site exploration for new projects.

A NRTK GNSS system can be integrated with the total station for instantaneous determination of the total station location by mounting the GNSS antenna directly on top of the total station alidade in open sites. Thus, the need for establishing permanent horizontal control stations onsite can be minimised. For orientation determination, the total station can be sighted at a back station, where its coordinates can be instantaneously determined using the NRTK-GNSS technique. This process improves the economics of surveying work, and reduces the overall surveying time, including the time required for the initialisation of the total station at each setup. However, one should note that performance of surveying with RTK GNSS in construction sites are affected by satellite availability, multipath errors resulting from working near buildings, and latency of the reference data. The influence of

the number of satellites in view, dilution of precision (DOP) and age of corrections over the accuracy and stability of the NRTK GPS solution was discussed in some studies, e.g. Aponte et al., 2009.

The performance of surveying with the network RTK approach in construction sites was evaluated by two tests. The first test was executed for checking performance in determination of planimetric coordinates and the second test for evaluation of performance in height determination. The first test was carried out during construction of a large building in Dubai for checking of positions of surveying marks of the footings, landscaping and the access road of the building. 48 points were used for checking purposes including 18 points on the boundary of the road, 19 points for the footings, and 11 points for the landscaping. These points were set out using a calibrated total station of 1 second precision. Next, point coordinates were computed from the working drawings and uploaded to a GPS controller, where a single GPS dual-frequency receiver was independently used for positioning of the test marks by utilizing data from the Dubai NRTK, known as Dubai Virtual Reference System (DVRS). The network consists of five continuously operating reference stations with baseline lengths ranging from 23.4 km to 90.8 km and uses the VRS algorithm. The position of each test point was determined after 10 seconds of data collection, which were recorded at 2 seconds interval. The shifts between the positions determined from the two methods, namely: calibrated total station and NRTK using observations from GPS, were measured and compared to represent the precision of the latter method if used in construction sites instead of the former method. Figure 4 illustrates the differences between the two methods. The statistics of coordinate differences between the two methods in easting and northing are given in Table 1. The average norm of the spatial differences ($\sqrt{(diff_E)^2 + (diff_N)^2}$) was generally less than 1.45 cm whereas the maximum difference was 3.45 cm. The small errors can be explained by the presence of one of the network reference stations within a few kilometres, which would be picked up by the system as the master reference station. Thus, most orbital and atmospheric errors were cancelled and the remaining errors would mainly be due to data noise and multipath. These results show that the GPS-RTK network approach can be used in the setting out of medium-accuracy surveying marks.

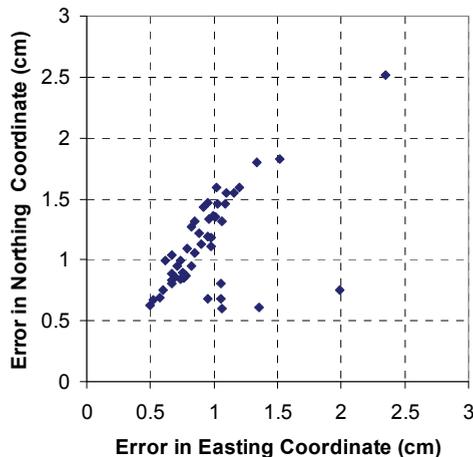


Fig. 4. Positioning differences between GPS Network RTK and the total station

	Average (cm)	Maximum (cm)	RMS (cm)
E	0.95	2.35	1.00
N	1.12	2.52	1.18

Table 1. Statistics of positioning differences between GPS RTK-Network with the total station

To check repeatability of results (internal accuracy), another independent survey was carried out after two days and 4 hours to re-determine the coordinates of the same marked points previously determined by GPS. The average and maximum values of the differences between the results of the two surveys are given in Table 2. As the table demonstrates, repeatability testing showed that the average value for differences in the total planimetric coordinate estimation was at the cm level, while for ellipsoidal height determination it was 1.56 cm. These differences can be attributed to changes in the quality of the measurements used, which mainly resulted from differences in the number of the observed satellites and their geometric distribution. These parameters have affected the quality of the network computations of the measurement corrections and the quality of coordinate estimation at the rover.

	Average (cm)	Maximum (cm)
E	0.85	1.23
N	1.15	1.88
h	1.56	3.22

Table 2. Statistics of coordinate discrepancies between different observing sessions

Unlike traditional levelling, GPS derived heights are referenced to an ellipsoidal datum (WGS 84) and do not depend on local gravity variations, whereas in most levelling works and mapping orthometric heights are used. Orthometric heights reflect changes in topography as well as local variations in gravity. They are referenced to the geoid, which is an equi-potential level surface of the Earth that is closely associated with the mean sea level on a global basis. To convert ellipsoidal heights from GPS (h_{GPS}) into orthometric heights (H), geoid heights are needed, such that:

$$H = h_{GPS} - N \quad (23)$$

where (N) is the geoid height. Thus, with the use of one receiver and employing NRTK to determine ellipsoidal heights, orthometric heights can be determined if a good geoid model is available.

To assess accuracy of orthometric height determination by using NRTK, the second test was performed in Dubai, using the DVRS network. The Dubai gravimetric geoid model was used, which was developed by integrating a comprehensive set of gravity measurements with GPS, levelling and digital elevation data. The computed geoid fits GPS/levelling at the 3-4 cm level RMS (Forsberg *et al.* 2001). The test was performed on a network consisting of 41 benchmarks of the second order levelling network. Orthometric heights at these

benchmarks were first estimated by combining ellipsoidal heights determined by using the Dubai NRTK with the local gravimetric geoid model data and were next compared to known orthometric heights of the benchmarks. The test area spanned approximately 22.7 km x 7.8 km in the Easting and Northing directions respectively, representing the area acquiring the most demanding survey works in the Emirate of Dubai. The height difference between the highest and lowest points in this test was approximately 34.5 meters. Each test point was occupied for a period of a few seconds, representing an ordinary working environment. The standard deviations of the ellipsoidal height determination for the occupied points of the test network ranged between 1.05 cm and 5.47 cm (El-Mowafy *et al.*, 2005).

Figure 5 shows the differences in orthometric heights between using the “NRTK GPS + geoid heights” and the known orthometric heights of the benchmarks. On average, differences were within ± 5 cm, with a maximum value of 7.04 cm. The statistical results of the differences are presented in Table 3. The average value of the absolute differences was 2.4 cm with 3.05 cm standard deviation. The differences towards the north-east were greater than those at the south-west region of the test. This can mainly be attributed to accuracy of the geoid model used in the test area. Figure 6 illustrates the surface plot of the height difference between the two methods. These results show that no significant systematic errors were present. The achieved accuracy is considered precise enough for third order levelling, which represents the majority of levelling works being carried out.

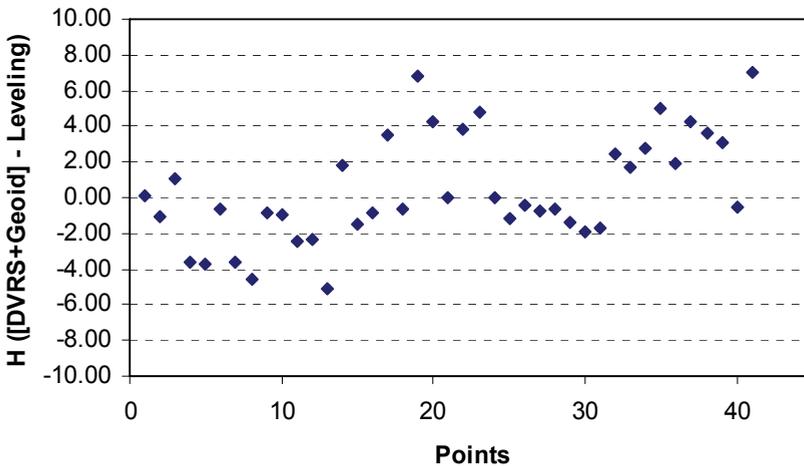


Fig. 5. Height differences between GPS Network RTK geoid heights and precise levelling (cm)

Average of absolute values	Max. difference	σ
2.40	7.04	3.05

Table 3. Statistics of height differences between the GPS network RTK + geoid and precise levelling (cm)

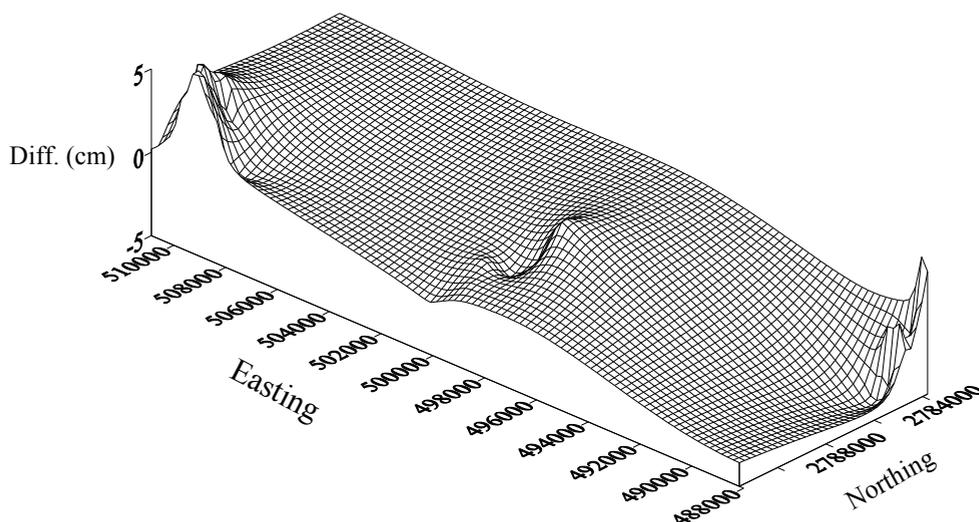


Fig. 6. Surface plot of the height differences

7.2 Using RTK GPS for remotely monitoring and controlling machine automation

The use of Real-time GNSS positioning for machine automation can enhance its productivity and functionality. Real-time GNSS positioning can provide cm accuracy, facilitating high performance and output for machines that require positioning data. The Supervisory Control and Data Acquisition (SCADA) is a good example for supporting a field automated system. Having real-time accurate GNSS positioning information as an input to the SCADA system gives a lot of opportunities to develop many solutions for planning, design, construction and monitoring field operations that require precise positioning.

An automated machine such as a field tractor, excavator, or a driller can be automatically operated, unmanned and fully remotely monitored and controlled. The machine can be controlled by the analogue and digital outputs from a Remote Terminal Unit (RTU). The RTU can have preloaded Programmable Logic Control (PLC) software that activates the outputs according to the field inputs from the machine primary sensors as well as the real-time accurate coordinates fed from a GNSS unit receiving measurement corrections from a NRTK centre. The field operations, events, logs and alarms can be fully remotely monitored through a SCADA system. For the SCADA system programming software, standards such as IEC1131-3 can be used, which is the international standard for controller programming languages. It specifies the syntax, semantics and display for the PLC programming languages. An open standard communication protocol such as Distributed Network Protocol (DNP) can be used. DNP is a set of standards and interoperable communication protocol used between companies in processing automation systems. These open source standards and interoperable platforms will allow the system design to be implemented in any of the industrial proven commercial SCADA systems.

The functionality and process automation of the system can be described in two scenarios. In the first scenario, the machine can be operated in a semi-automated mode with online

telemetric control. The real-time GNSS positioning is computed and a Geographic Information System (GIS) is utilised at a control centre that operates the machine in a telemetric mode. In the second scenario, the system can be fully automated based on pre-set instructions and automated integration with GIS planning, database and geo-coded maps. In this case, the GIS system is uploaded in the field machine.

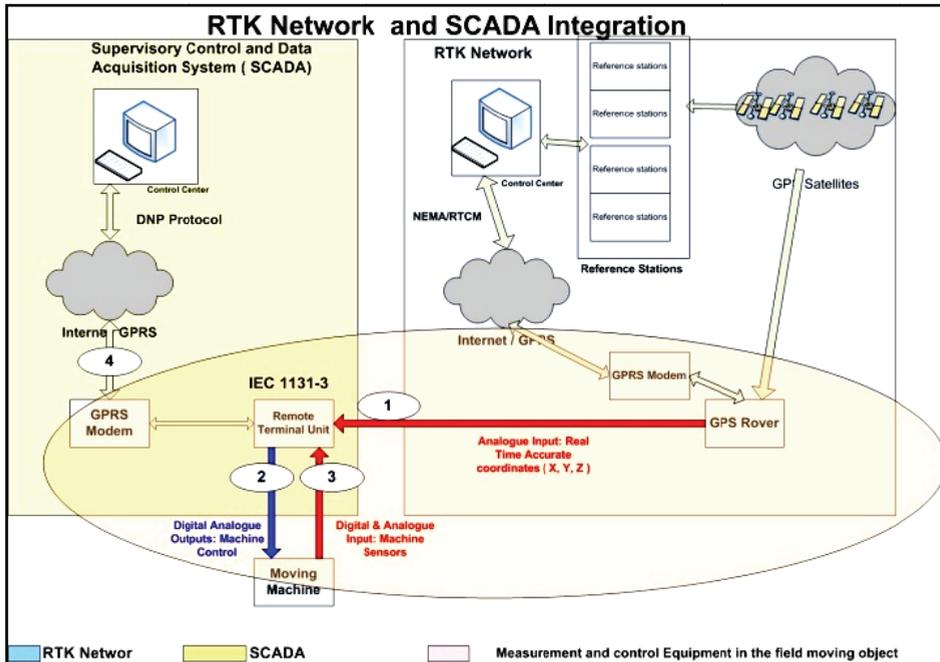


Fig. 7. Proposed network RTK GNSS and SCADA integration

Figure 7 illustrates a developed architecture for the first scenario, where the machine is remotely controlled from a SCADA centre. In this case, SCADA monitoring and controlling hardware and software can be automated in the field with a built-in computer that has all required software and is placed inside the machine. The SCADA system can be integrated with the GIS through an interface where the mimics are truly geo-referenced maps representing the reality of the field in a full 3D mode. The GNSS and the primary sensors are sent through the RTU to the SCADA system to update the GIS map with real-time information. The following procedure can be applied (El-Mowafy and Al-Musawa, 2009):

1. The roving GNSS will be mounted on the field machine and sends its observations to the network centre. Positioning information is computed at the centre for telemetric control of the machine.
2. The control centre operates the SCADA monitoring and controlling hardware and software. The field automated process is fully monitored and controlled in a remote mode. Report, alarms, trends and historical records can be retrieved at the centre.
3. SCADA system chooses the planned field works based on the 3D accurate coordinates of the field points and using GIS.

4. The centre sends primary information to the field RTU unit that controls the machine. These commands can be either full machine commands or only the main commands that transfer between phases of field operations if detailed information is pre-fed to the RTU.
5. The RTU based on a preloaded automation process program will produce the controlling outputs based on a set of variable inputs from the primary sensors as well as the input of the position information received from the centre.
6. Real-time data from the GNSS and the primary sensors are sent remotely from the field through the RTU to the SCADA system to update it with the actual work progress in a feedback loop.

The logical sequence of the soft PLC process program of the RTU is illustrated in Figure 8 as a flow chart. As the figure depicts, the program starts with initialising and testing the system availability and health status. It reads the current positioning information of the machine as well as the status of the inputs from primary sensors. It executes the RTU process program while monitoring any alarms or malfunction signals for an emergency stop. The RTU events, logs and alarms are sent to the remote SCADA centre.

The machine automation in this scenario can be fully automated in the field. In this case, the GNSS unit mounted on the field machine receives the corrections from the NRTK and continuously feeds the RTU with the real-time positioning information representing the exact location of the machine in the field. The automation system is uploaded on a computer fitted in the machine and all control is pre-programmed and work is executed online in a feedback loop to keep up with the pre-set design. The real-time 3D accurate coordinates of the field points are to be input to a GIS system on board, where its output is fused with the PLC program. Work orders can be downloaded remotely to the field RTU at any stage if a change of plans is required. The RTU can also be connected through a modem to a remote monitoring centre with the ability to control and adjust the field process based on any new input. The field automated process can thus be fully monitored and controlled in a remote mode. Report, alarms, trends and historical records can be archived at the centre.

To investigate positioning precision that can be obtained from network RTK for machine automation, a test was performed in Abu Dhabi. The Abu Dhabi network RTK was used in this test. The network consists of 20 reference stations with separating distances between stations ranging between 60 km to 209 km. Several types of NRTK techniques can be implemented as per user choice, including the VRS method, the MAC approach, the FKP, and standard RTK using a single nearby reference station. The proposed approach was tested in the marine mode for one hour where a Leica 1200 GPS system dual-frequency receiver was mounted on a dredger working in a small island close to Abu Dhabi main island. The NRTK positioning system was operating at a sampling rate of 1 Hz. In addition, the data were internally stored for post-mission processing to act as a reference for comparison with network RTK results to assess its accuracy for the application being tested. The rover data were referenced in this case to station ADCC of the Abu Dhabi continuously operating reference network. The distance between this station and the test trajectory was 6 km on average, giving stable ambiguity fixing with precise positioning output. When comparing the two sets of positioning results (Network RTK and post-mission processing) the differences were at the cm range. The average precision of the determined positions in NRTK mode was 2.85 cm for the horizontal components and 4.1 cm for the height. Statistics

of the positioning differences between the two methods are given in Table 4. During testing, availability of NRTK was higher than 95%. These results show that the NRTK can be successfully used for positioning of field machines.

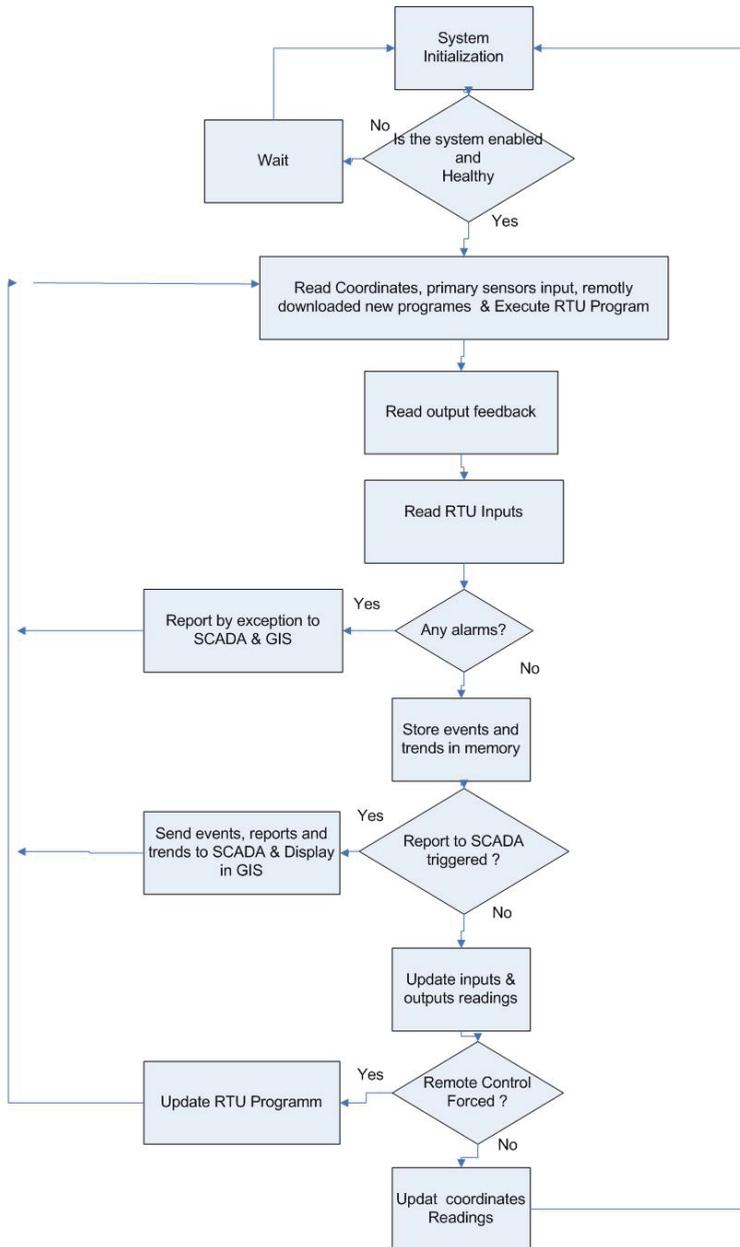


Fig. 8. Flowchart of the RTK GNSS & SCADA/GIS Logic control

S.D	Maximum (cm)	Average (cm)
σ_E	3.60	1.92
σ_N	3.93	2.11
σ_h	9.03	4.10

Table 4. Difference between network RTK and post-mission positioning in machine automation testing

7.3 Using network RTK in the airborne mode

The network RTK approach is mostly used in static or kinematic ground applications. In this section, the use of the NRTK approach in the airborne mode is discussed. At present, positioning by GNSS is a widely used technique in the airborne mode for geo-referencing of aerial mapping data and surveillance by Unmanned Aerial Vehicles (UAV). In aviation, it is estimated that from 2015, most new commercial aircraft will be fitted with GNSS to enhance precise navigation and make it safer (Pedreira, 2009). However, at the moment, GPS is the only approved system as a stand-alone aid for non-precision approaches (Radišić, 2010), e.g. as a supplementary navigation system and for positioning in non safety-of-life applications. This is mainly due to the need to achieve high level of performance in terms of integrity, availability and reliability in the airborne navigation, which GPS on its own cannot reach due to the limited number of satellites available in one site at any particular instance. This situation is expected to improve with the addition of the new systems such as Galileo and Compass. When using network RTK in the airborne navigation, additional concerns have to be addressed, which include:

- Due to the high dynamics involved in the airborne navigation, a high update rate of sending the corrections is needed compared with the rate implemented for land applications. This rate has a direct impact on the Time-To-First-Fix of phase ambiguities, and thus on the overall positioning feasibility and accuracy (El-Mowafy, 2004).
- The format of GPS measurement corrections should be standardised to ensure that the system is independent of any single receiver manufacturer. The use of the RTCM Version 3.1 standards is thus recommended.

The main advantages of using network RTK for precise airborne positioning can be summarised as follows:

- No dedicated ground reference stations are needed for post-mission or real-time applications.
- Unlike standard differential positioning, the distance between the aircraft receiver and the nearest reference station does not present a concern as long as the aircraft flies within the network RTK area of coverage.
- In navigation, due to the fact that networks RTK usually have an area of coverage that extends to several hundreds of kilometres, each network can cover more than one airport, including small airports, unlike the current Local Area Augmentation systems

(LAAS) implemented only in some major airports. NRTK systems can also be used in search and rescue operations, emergency landing, road traffic monitoring from the air, as well as emergency response.

- Compared to LAAS, no significant additional infrastructure cost is involved as the hardware and software of the GNSS-NRTK are available in most developed countries and the establishment of new networks is currently underway or planned in different regions worldwide.
- Network RTK provides cm to decimetre positioning accuracy even in the case of malfunctioning of some reference stations, particularly for dense networks.
- Network RTK can give better runway utilisation by improving airport surface navigation. It can also enhance air traffic management by increasing dynamic flight planning.

The use of the VRS technique in the airborne mode is not generally recommended since in this high velocity environment continuously updated approximate coordinates have to be used for the VRS computation. This is similar to having a moving reference station. A system reset should thus be frequently performed when the VRS coordinates are changing, which will result in frequent initialisation of the carrier-phase ambiguities. Therefore, it is preferable to keep the VRS location for the longest possible range. An alternative approach would be to apply the PRS technique, where the PRS points are chosen along the path of the final approach and close to and at the airport. Furthermore, the duplex communication mode used in the VRS technique is limited by the ability of the processing centre to simultaneously perform calculations for all users. As this number grows, extended latency in receiving the corrections may result. Additionally, the possibility of signal breaks in the duplex communication mode is more than the case of using a one-direction communication. Thus, the use of a one-directional communication method, e.g. applying the FKP method, would be more appropriate for the airborne mode. The PRS and Mac techniques can also be implemented in the one-directional mode, whereby the PRS or the Master-Auxiliary stations are selected to cover a specific area, such as the airport. The establishment of ground transmitters at the airport can improve availability of the corrections.

The feasibility of using real-time reference networks for positioning in the airborne mode was examined using the DVRS NRTK over the city of Dubai. Flight tests using a helicopter and a small fixed-wing airplane were carried out. The trajectory of the fixed-wing aircraft test is illustrated in Figure 9. The main parameters under investigation were the achievable accuracy and availability of VRS measurements. In these tests, aircraft positions were determined using a dual-frequency GPS receiver (Leica SR530). The data were processed in real time at one-second intervals. The DVRS reference stations collect and process data at five-second intervals. Thus, the NRTK data were interpolated in time for the rover receiver to compute positions at the one-second interval.

To assess performance of NRTK approach for this test, the results were compared with positions determined from a standard double-difference technique whereby the observations of the aircraft receiver were stored and processed in a post-mission mode. The aircraft data in this case were referenced to one of the DVRS network stations located within a range of a few kilometres from the flight route. Precise IGS orbits were used in the post-mission processing. The differences between the two methods (NRTK and post-mission processing) are given in Figure 10. For the test at hand, the DVRS data were lost for some periods, which ranged from a few seconds to three minutes. The periods when the DVRS

data were available are shown in the dashed areas in Figure 10. The temporary break in reception of the NRTK data can be attributed to the use of GSM signals as the means of communication between the DVRS centre and the aircraft at the time of the test. However, the GSM signals were only used for testing purposes. In practice, the problem of breaks in receiving the network corrections can be significantly alleviated by using more robust means of delivering NRTK service to the aircraft.

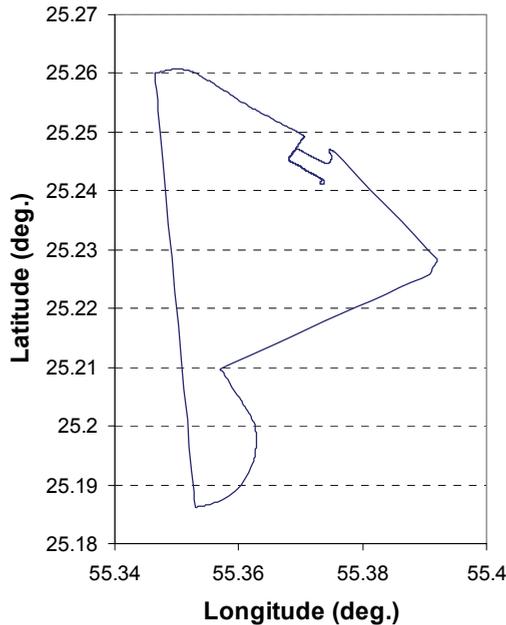


Fig. 9. Trajectory of a fixed-wing aircraft test

When comparing the results of positioning obtained by the DVRS NRTK with the post-mission double-difference positioning for the periods where NRTK data were received and phase ambiguities were fixed, the average 2-D and height positioning discrepancies between the two methods were at a few cm level, as they were 1.6 cm and 2.8 cm respectively. The differences can mainly be attributed to the model assumptions and procedure in the two techniques. During the period when phase ambiguities were only solved in a float solution, the differences were 26.3 cm and 52.5 cm. However, when the DVRS NRTK data were lost, positioning accuracy deteriorated to the metre level. In this case, supporting methods, such as good prediction algorithms and integration with other sensors, e.g. a geodetic-grade inertial system, are needed to cover the short periods when breaks in reception of the measurement corrections take place. Several methods for prediction of NRTK observation corrections as a time series were investigated in El-Mowafy, 2008. Different time-series prediction methods were investigated for different types of errors. The double exponential smoothing prediction approach performed best in most of the cases when studying the satellite clock error corrections. Winters' method and the Autoregressive Integrated Moving Average (ARIMA) model were the best methods for predicting the orbital and wet tropospheric errors, respectively.

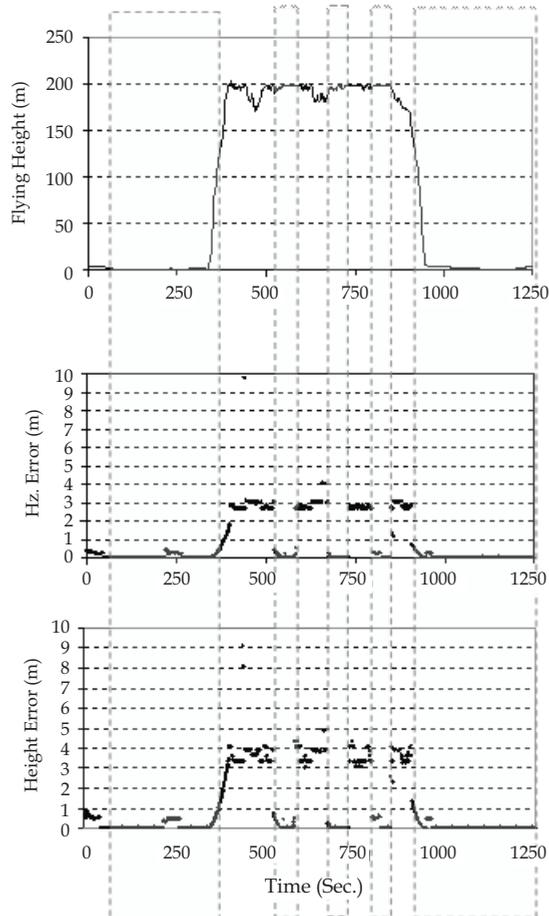


Fig. 10. Fixed-wing aircraft test results

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