Surveying with GPS for Construction Works Using the National RTK Reference Network and Precise Geoid Models

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Key words: GPS, Engineering surveying, Wide area networks, Positioning

SUMMARY

The paper discusses surveying by means of RTK-GPS positioning techniques in construction work with emphasis on using a single GPS receiver. The use of the traditional RTK GPS method is first discussed, followed by outlining the principles and advantages of using RTK reference networks. Possible applications of these techniques in the construction field are given. A system consisting of a triad of three cheap orthogonal laser pointers is presented for rapid setting out of surveying marks and for minimizing the number of points occupied by GPS during night work. The system can be used to set out several points along and perpendicular to any line on the site. A model is also presented for transferring map distances derived by the GPS grid coordinates to their respective ground distances. The presented techniques were evaluated during construction of a large building and its access road. Measurement corrections were employed from a national RTK network in Dubai Emirate, UAE. The RTK GPS method was also used for determination of instantaneous orthometric heights from GPS ellipsoidal heights and geoid heights. The latter were interpolated from a recently developed precise gravimetric geoid model for Dubai. In addition, the GPS was used for instantaneous determination of a total station location working on the site, by mounting the antenna directly on top of the total station alidade; thus eliminating the need for establishing permanent horizontal control stations. Results showed that positioning accuracy of 1-3 cm was generally achieved. Repeatability results showed, however, that cm-level differences in coordinate estimation can still be experienced between different observing sessions. Thus, this method is appropriate for medium accuracy construction surveys, such as grading and staking out of road marks, footings, pipelines, utilities, landscapes and fences.

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1. INTRODUCTION

Surveying work needed on construction sites is usually dependent on the determination of accurate plane coordinates and heights. With current developments in the Global Navigation Satellite Systems (GNSS), real-time 3D positioning can be generally achieved at the cm level of accuracy. Such accuracy helps to expand the use of Real Time Kinematic (RTK) GPS positioning techniques for construction surveying work, particularly for large sites when rapid survey is needed. Early studies by Sumpter and Asher (1994) on using RTK techniques in the setting out of cadastral survey marks estimated cost savings of 25-50% compared with ground survey methods. This ratio has been improving over the years with improvements in software and hardware capabilities. The reduction in field expenses results from the fact that in setting out marks by means of RTK GPS, the number of surveying crew members can be reduced, frequent setups of the surveying instruments are no longer required, and the need for accurate local traverses or multiple control stations within the site is eliminated. Cost savings do not come, however, with a significant loss in accuracy, and previous studies showed that RTK GPS and the traditional techniques employing total stations give statistically compatible results (El-Mowafy, 2000). In addition, with the implementation of the national RTK network of reference stations, the user no longer needs to establish his own GPS reference station on site, as the required measurement corrections can be received from the existing continuously running network. This approach is expected to dominate surveying by GPS in the construction sector due to reductions in field personnel and cost, and the increase in versatility and reliability.

2. USING THE CLASSIC RTK TECHNIQUE IN CONSTRUCTION SURVEY WORKS

In the differential RTK GPS positioning approach, the position of the rover is accurately determined at the cm level in real time using information from a reference station of known coordinates. Positioning can be carried out either by differencing the measurements of the rover station with measurements received on-line from the reference station, or more commonly by computing range corrections at the reference station and applying them on-line at the rover. A range correction can be estimated for each satellite as the difference between the measured range and the correct one, which can be estimated from the coordinates of the known satellite and the reference point. Details on the process of estimating phase-measurement corrections and their format can be found in Langley (1998).

In the RTK technique, positioning latency, which is the time needed for transmitting the data from the reference station to the remote receiver and processing it, is expected. Latency time can range from 0.1 second to a few seconds, depending on the amount of data sent (data format), type of data link used, reference-to-remote distance, and the measuring conditions. Two techniques can be applied to reduce degradation of accuracy due to latency. The first is the synchronized approach, and the second is the rapid approach. In setting out applications, the synchronized approach can be used, in which data are stored, time tagged and processed once the corrections of the same epoch sent from the reference station arrives at the rover. The accuracy of this technique does not degrade significantly due to latency, and cm-level accuracy is usually achieved. Although the coordinates of a certain epoch are computed a few seconds after their collection, occupying the same point for a few seconds usually resolves this problem. On the other hand, in the rapid approach, a prediction algorithm is applied to estimate measurements from the received observations, which are differentially used with the rover data once they are collected. Some positioning errors are thus expected depending on the dynamics of movement. The rapid approach is suitable for on-line applications, such as grading or checking of the as-built utilities with their design.

To maximize the area covered by RTK, the antenna of the reference station should be placed in locations free from multipath and mounted as high as possible. Also, higher data transmission rates are needed if high accuracy is required. A baud rate of 9600-19200 bps is currently used. The UHF and spread spectrum radio modems are the most popular for RTK applications. In optimal cases, spread spectrum radios (2.4 GHz) can have a range of about 20 Km with a 35-Watt amplifier. UHF has an even longer range. This range is dependent on the power of the transmitting radio, the interference environment, the terrain geometry, antenna setup, and structure density of the working area. Terrain blockage in particular can result in the loss of radio link. In some situations, placement of radio-modem repeaters can help, particularly in establishing long roads. The repeaters can be mobile, moving between locations within the site according to the progress of the work. The mobile repeater can be mounted on a survey truck or similar.

One should however consider that short ranges are usually preferred in order to minimize the ionospheric error, which is a main concern in determining ‘fixed’ carrier-phase ambiguities, improving positioning accuracy as well as reducing latency. Thus, the distance between the two receivers should be kept to within about 10 Km (El-Mowafy, 2000). If long base-to-rover ranges are needed in large-area projects (e.g. in establishing new highways or in their maintenance), more efficient models of spatially correlated errors will be needed. This concept is implemented in the network approach.

3. THE RTK NETWORK APPROACH

The constraint of limited reference-to-rover range can be removed by using a network of reference stations, whereby observations from multiple reference stations are gathered and processed in a common network adjustment at a central processing facility. The distances between the reference stations are usually selected to be less than 100 km in order to achieve a fast and reliable ambiguity resolution between the reference station vectors. The main advantages of this method over the traditional RTK approach include: cost-reduction, reduction in number of staff, and the obtaining of consistent coordinates. The communications between the reference stations and the control center are usually carried out via dedicated telephone lines. Measurement corrections at the reference stations are computed at the central station. Data are sent to the rover, such that either the rover or the central station interpolates a set of corrections at the rover location or at a virtual reference station close to it. These corrections are then used to correct rover observations in real time to accurately determine its position, see for instance Eng et al. (2000) and Jansen et al. (2002). The network solution works on reducing bias sources, particularly distance dependent errors, such that the distance between the rover and the closest reference station can be extended to several tens of kilometers.

A minimum of three reference stations are required to generate RTK network corrections. However, at the national level this number should increase to cover larger areas. As this number increases, redundancy increases, and better corrections can be estimated. If reference stations have a temporary malfunction (e.g. due to loss of power), 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).

Different network algorithms can be applied for generating measurement corrections at the reference stations and interpolating them to the user. Among these are: the virtual reference station method (VRS), the Multiref method (conditional adjustment method), the correction grid method, the correction function method, and the area correction parameters technique (FKP), see for instance Raquet and Lachapelle (2001) and Wübbena et al. (2001). A mix of these techniques is also possible. The difference between the results of the different techniques is not significant and generally lies within 10% (Petrovisky et al., 2001). A comparison between the main methods can be found in El-Mowafy (2004). Among the different models presented for interpolation of the corrections, the plane surface technique has so far given the best results in modeling the regional trends of the correction differences (Euler et al., 2004). A distance-based low-order surface model using a bilinear polynomial can be used for this purpose. For instance, it can take the form:

$$f\,(E, N) = a\,(E - E_o) + b\,(N - N_o) + c \tag{1}$$

where (E, N) and (E_o, N_o) denote the easting and northing coordinates of the interpolation point and the origin, respectively. a, b and c are the coefficients of the plane model, which can be estimated from a weighted least squares solution from range residuals at each reference station of the network.

4. APPLICATIONS OF REAL-TIME POSITIONING BY GPS IN CONSTRUCTION WORK

To obtain real-time positioning accuracy at the cm level, the carrier-phase measurements should be employed after resolving the initial integer number of cycles (phase ambiguity). For real time applications, the main elements affecting the performance of the RTK GPS surveys in the construction field are: satellite availability, multipath errors resulting from near large buildings, and latency of the reference data. Satellite availability can change according

to the time of measurement, and as a result of partial or full sky blockage when working close to buildings or trees.
With a typical accuracy of 1-5 cm, the RTK GPS technique can be utilized for construction survey work such as:

- grading,
- elevation determination during instillation of utilities (e.g. pipelines, power lines, cables),
- staking out of road marks, footings, pipelines, utilities, landscapes, fences etc.,
- mapping,
- checking of the as-built with the designs,
- site exploration for new projects.

The RTK GPS system can be integrated with the total station, such that it can be used for instantaneous determination of the total station location by mounting the GPS antenna directly on top of the total station alidade in open sites; thus eliminating the need for establishing permanent horizontal control stations on site. For orientation determination, the total station can be sighted at a nearby point (back station), where its coordinates can be instantaneously determined using the RTK-GPS technique. This process improves the economics of surveying work, and reduces the overall surveying time, including the time required for the initialization of the total station at each setup for vertical plummeting over a known point.

5. USING GPS-DERIVED COORDINATES, HEIGHTS AND DISTANCES IN CONSTRUCTION WORKS

The primary positioning output by global navigation satellite systems, e.g. GPS, is point Cartesian coordinates in an earth fixed frame, such as the WGS-84. These coordinates are usually transferred to the local-grid system to be used in construction work. For simplicity and consistency of results, the UTM projection referenced to the ITRF frame can be employed. Heights, on the other hand, are generated by GPS in an ellipsoidal form referenced to the WGS-84 datum. To relate these heights to the ones used in construction sites, which are usually referenced to the mean sea level (orthometric heights), accurate local geoid models should be used. The resulting orthometric heights can be computed from the relation:

$$H = h - N^* \tag{2}$$

In the case that no accurate local geoid model exists and the survey work is carried out in a limited area, orthometeric heights can be estimated from:

$$H = H_{reference} + \Delta h - \Delta N^* \tag{3}$$

where:

H	orthometric height at the unknown point
h	ellipsoidal height at the unknown point
N*	geoidal height

$H_{reference}$	orthometric height at the reference point
Δh	differential GPS-derived ellipsoidal heights relative to the local datum
ΔN^*	differential geoid heights

Due to the fact that geoid heights have a little variation for small areas, the last term in equation (3) can be ignored. In this case, the resulting error due to such approximation is expected to be in the range of a few millimeters. Geoid heights can also be approximated using a global model such as the EGM96, which does not address the local changes of the geoid. As a result, some accuracy will be lost in this case, particularly for long lines.

In the setting out of surveying marks, maps used are usually given in plane coordinates with distances referred to the earth's surface. On the other hand, maps based on GPS measurements have points of plane-grid coordinates (East, North, and height) transformed from geographic (ϕ, λ, h) coordinates using a map projection technique. As a result, there will always be a difference between distances measured in the field and their corresponding distances estimated from the map between points determined from their GPS grid coordinates. Such a problem is usually encountered when checking distances between staked marks as a method to verify their correct locations. The surface distance between two points (e.g. 1 & 2) should not then be measured from the map; instead, it should be computed from their plane coordinates. *Figure 1* gives a flowchart demonstrating the precise steps of this process. The interested reader may refer to USGS (1994) and Brinker and Minnick (1995) for details concerning the mathematical modeling involved. As *Figure 1* shows, the main data required are the local datum parameters and its relation with respect to the map projection system used. Geoidal heights are also required if orthometric heights are needed. In the field, the surveyor has to have a programmable pocket calculator or a notebook computer to perform the computations required.

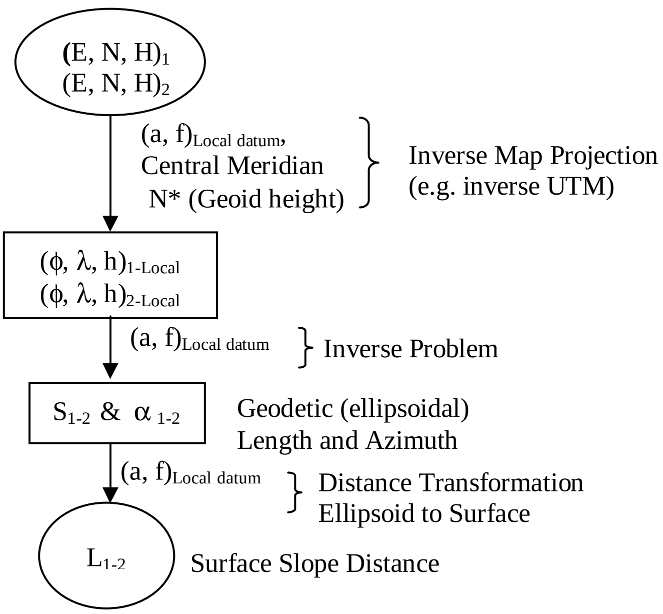

Fig. 1: Computation of the Surface Distance from the Plane Coordinates of its Points

For simplicity, if the plane (grid) distance is used, its transformed surface distance can be deduced from the following equation:

$$L_c = S\ [\ L_e^2\ (1 + \frac{h_1}{R_\alpha})\ (1 + \frac{h_2}{R_\alpha}) + (h_2 - h_1)\ ]^{1/2} \qquad (4)$$

where L_c and L_e denote the surface slope and grid distances, respectively. S accounts for the scale factor in map projection in addition to the relation between the ellipsoidal chord distance and the grid distance. R_α is the normal section radius of curvature computed for the azimuth (α) of the line 1-2, and h_1 and h_2 denote the ellipsoidal heights at the two points computed in the local ellipsoid.

6. AIDING SETTING OUT WORK USING A LASER POINTING SYSTEM

To speed up the staking process by minimizing the number of points occupied by GPS, a system is developed consisting mainly of three cheap orthogonal laser pointers. The three laser pointers are aligned to generate three perpendicular laser beams, see *Figure 2*. The three pointers are held by a special frame that can be mounted on a tribrach. When the system is leveled by the base screws, one pointer heads down for plummeting of the laser system. For densification of surveying marks along a desired direction, the second laser pointer is sighted at a mark on that direction. Then by unclamping the pointer, it can be rotated in the vertical plane to set out points along the desired direction. The third laser pointer can be used to mark points in a direction that is perpendicular to the first one. This laser pointer has also the freedom to rotate in a vertical plane to set out points at variable distances. This system can be used to provide a fast method for setting out several points along and perpendicular to any line in the site replacing the traditional optical instruments. The system is particularly helpful for setting out points within short ranges during night work, or during periods of low brightness level of the sun at day time. It is also suitable for indoor use where GPS cannot be employed. However, the system cannot usually be employed in bright sunlight, as in this case, it will be hard to identify the laser point.

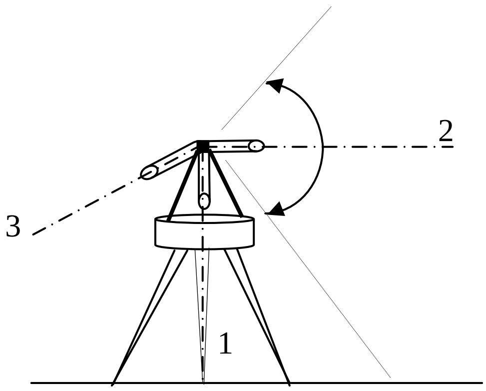

Fig. 2: A Developed Laser Pointing System for Setting out Applications

To calibrate the system, and particularly to fix the perpendicularity of its three laser beams, a calibrated total station was used. First, its telescope was leveled. Next, the total station was used to identify two points along its line of sight. The points were marked on sticks, fixed at large distances from the total station. The process was repeated for a second line perpendicular to the first one. Following that, the laser pointer system was set up on the same tripod, and its height was adjusted so that the center point of the laser triad occupied almost the same center point of the total station. The laser pointers were rotated so that they pointed at the points marked by the total station. The process was repeated with points of variable distances to gradually enhance perpendicularity of the laser pointing system until it was satisfactorily reached.

During the process of staking the surveying marks, miss-plummeting of the GPS antenna above the ground points required for staking is a common problem, as it is very hard, even for a well trained surveyor, to hold the rod carrying the GPS antenna vertically. This error can produce a positioning error of a few millimeters and can even reach several centimeters. To precisely plummet the GPS antenna, a special rod was designed employing laser technology. The developed device consists of an inverted L-shaped rod as depicted in *Figure 3*. Its vertical post is a telescopic rod with the capability of changing its height. The antenna can be clamped to the short horizontal bar. Attached to the horizontal bar, underneath the antenna, is a cheap laser pointer, such that the pointer center is collinear with the antenna center. This device emits laser vertically only when it is leveled, similar to laser levels. Thus, when plummeting the antenna, the ground point identified by the laser point should have the same plane coordinates as the GPS antenna. The surveyor can then accurately stake the surveying mark on the laser point. The short horizontal bar between the vertical laser beam and the vertical rod gives the surveyor enough space to carry out this task.

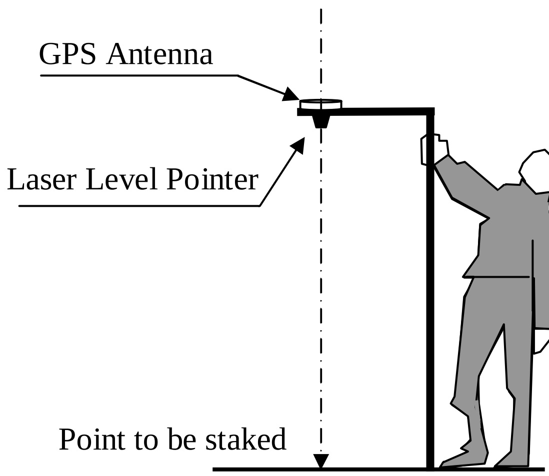

Fig. 3: Staking by Laser Plummeting at the Required Position

7. EVALUATING RTK ACCURACY FOR CONSTRUCTION SURVEY WORK

To evaluate surveying on construction sites using the network RTK approach, a test was conducted in Dubai, UAE, in which surveying operations were carried out using a single GPS receiver. The receiver employed, in real-time, data from the Dubai Virtual Reference Network System (DVRS), and the Dubai accurate geoid model. The DVRS network consists of five active reference stations, with baseline lengths varying from 23.4 km to 90.8 km. Communications between the user and the computing center are carried out in a duplex mode, in which the rover receiver sends its approximate position via a cellular modem, and receives the computed network measurement corrections. These corrections are lumped into one value for each satellite measurement and interpolated for a virtual reference station (VRS) close to the rover position. The raw data can also be reconstructed for each satellite at the VRS, and the corrections are applied to the calculated raw data. The rover can thus apply a standard differential approach with the VRS station to estimate its position, or determine it directly in a point positioning mode after correcting its measurements using the transmitted VRS corrections. The Dubai geoid model was developed in 2001 from varying data sources, including mainly gravity data, a digital elevation model (DEM), orthometric heights and GPS-observations at leveling benchmarks. Previous testing showed that differences in estimating orthometric heights between using the GPS ellipsoidal heights with precise Dubai geoid data and using spirit leveling were in the range of ± 4 cm (El-Mowafy et al., 2004).

7.1 Test Description

The performance of surveying with the network RTK approach on construction sites was evaluated during construction of a large building, its landscape and access road. Part of the test site is illustrated in *Figure 4.* The test included the checking of elevations on the site and the setting out of marks for the building footings and the access road. The total number of points used for height checking was 64 points, while the total number of the tested points for setting out was 48 points, including: 18 points for the road, 19 points for the footings, and 11 points for the landscape. Point plane coordinates and heights were first determined from the working drawings. These coordinates were next uploaded to the GPS controller. During testing, a single GPS receiver was used for positioning. The receiver accepted corrections from the DVRS network and used the Dubai geoid model to determine plane coordinates and heights. The DVRS data were received every five seconds, and thus were interpolated during positioning, as a positioning rate of one second was used. Throughout testing, the number of tracked GPS satellites and their distribution were generally “normal”, with 6 to 9 satellites being observed, and with PDOP ranging between 2.5 and 4.7. The GPS setting out software utility was used for directing the surveyor (by direction and distance) to the correct point positions, and the points were staked using the developed laser-aided antenna-plummeting rod.

To assess height determination with the GPS RTK-network approach, point heights that had been estimated by GPS were re-determined by spirit leveling. In the latter method, the reference benchmark used had its elevation determined by GPS to ensure consistency of results. On the other hand, to evaluate the RTK-network approach accuracy in setting out all test points, their locations were re-computed independently using a calibrated total station (Sokkia set 3000). The coordinates of the staked points were compared to their values derived from the working drawings. The total station occupied a central point in the site, which had its coordinates determined by a static GPS survey. The static GPS survey was also used to determine the coordinates of a second point, used as the back station for estimation of the initial azimuth.

Fig. 4: Part of the Test Site

In general, the main elements noted during testing that affect the performance of surveying with RTK GPS in the test site were: satellite availability, multipath errors resulting from working near buildings, and latency of the reference data. To minimize the impact of the first two factors in order to obtain reliable results, it was advisable to avoid working with the RTK GPS system close to existing nearby buildings wherever possible. Other methods, such as the traditional optical methods, had to be used for these locations.

7.2 Test Results

Figure 5 illustrates differences in plane-coordinates of the staked points using the GPS RTK-network technique and their values determined by the total station. The statistics of coordinate differences in easting and northing are given in the first two rows of *Table 1*, which include the average differences for all points and their maximum values. The average differences were generally less than 1.45 cm for the total of the 2D plane coordinates, whereas the maximum error was 3.45 cm. In another external test, the surface distances between the consecutive points staked by the RTK GPS survey were measured and checked against their correct values, computed from the map using the points grid coordinates. Differences in these distances ranged from 0.23 cm to 5.89 cm. These results show that the GPS-RTK network approach can be used in the setting out of medium-accuracy applications such as establishing roads, footing markers, boundary, utilities (pipelines and power lines), and landscaping work (parks, parking areas, walkways, etc.). The last column of Table 1 gives the RMS of the differences, which in general was also less than 1.55 cm for the total planimeteric error. Differences in height estimation between using GPS RTK-network approach with the precise local geoid data and the spirit leveling method are illustrated in *Figure 6*. The average and maximum values of these differences and their RMS are given in the last row of Table 1, which shows that the maximum error was less than 5 cm, while it was 2.31 cm on average. The proposed method of using the GPS RTK-network positioning technique and the local geoid data for determination of heights is thus suitable for several applications in the construction field, including: grading, laying out of utilities, and checking of elevations in landscaping.

Table 1: Statistics of Positioning Errors Using GPS RTK-Network and Local Geoid

	Average (cm)	Maximum (cm)	RMS (cm)
E	0.95	2.35	1.00
N	1.12	2.52	1.18
H	2.31	4.97	2.45

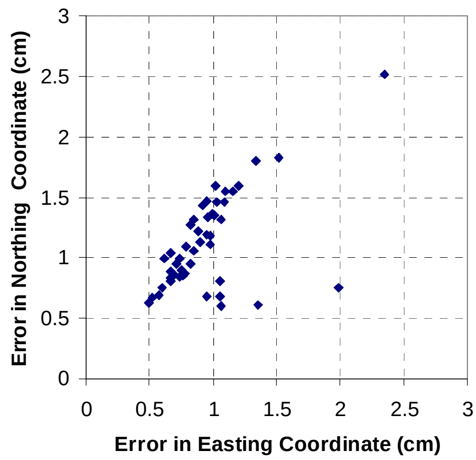

Fig. 5: Positioning Errors in Setting Out with the GPS RTK-Network Technique

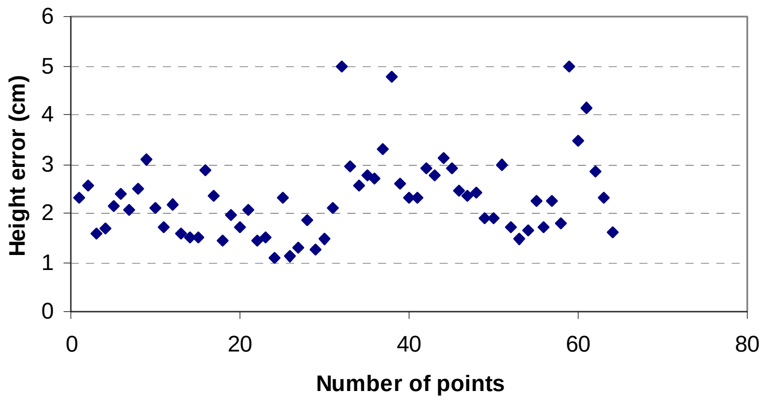

Fig. 6: Orthometric-Height Errors Using RTK-Network GPS + Precise Local Geoid

To check repeatability of results (internal accuracy), another independent survey was carried out after two days to re-determine the coordinates of the same points set out by GPS, or heights for the points that had height determination using the GPS RTK-network approach with the precise local geoid. The average and maximum values of the differences between the results of the two surveys are given in *Table 2*. As can be seen from the table, repeatability testing showed that the average value for differences in the total planimeteric coordinate estimation was 1.54 cm, while for height determination it was 2.49 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. This affected the quality of the network computations of the measurement corrections, and the quality of coordinate estimation at the rover.

Table 2: Statistics of Coordinate Discrepancies between Different Observing Sessions

	Average (cm)	Maximum (cm)
E	0.75	3.23
N	1.35	2.55
H	2.49	5.48

The developed laser pointing system used for the densification of points between the points staked by the RTK GPS was next tested. The laser pointer was used in one setup to set out 12 equi-spaced points, at intervals of 5 meters, between four consecutive points staked by the RTK GPS technique. When sighting by the calibrated total station between the closest and farthest points, a deviation of the laser beam was recorded with a maximum value of 9 millimeters. One notable drawback of the laser pointer used was the increase of its diameter to a few millimeters as a function of distance.

8. SUMMARY AND CONCLUSIONS

Determination of 3-D positions required in construction work can be achieved using a single GPS receiver employing the RTK network approach and a precise local geoid model. Setting out by RTK GPS is a fast and cost saving method that can be applied for medium-accuracy applications accepting positioning errors at the cm level, where less than 3.5 cm accuracy in plane coordinates can be routinely achieved. This is suitable for the staking out of road marks, footings, pipelines, utilities, landscapes, fences etc. An accuracy of less than 5 cm can also be achieved for height determination using the RTK network approach, where orthometric heights compatible with spirit leveling heights were determined using the precise local geoid model. Such accuracy is suitable for grading, landscaping, installing utilities such as pipelines, power lines and power cables, as well as checking the as-built structures with designs. For both planimeteric and height estimation by GPS, discrepancies in coordinate estimation can be experienced between different observing sessions within a few centimeters.

Setting out can be accelerated at night or away from direct sunlight by using a system of three orthogonal laser pointers. The system can be used to stake several points within short ranges along and perpendicular to any line in the site, replacing the traditional optical methods. The common problem of miss-plummeting of the GPS antenna above ground points while staking can be resolved by using a specially designed inverted L-shaped rod supplied with a vertical laser pointer collinear with the antenna centre. The projection of the antenna center on the ground is visualized by the laser point, at which a mark can be staked.

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