ON THE PERFORMANCE OF GNSS LEVELLING OVER STEEP SLOPES

Sobre o desempenho do GNSS no nivelamento de declives acentuados

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ABSTRACT

In geodetic applications variety, one of the main current focuses is recently to determine the heights of ground stations with high accuracy. Specially the possibility of acquiring 3D information of the point positioning with high accuracy is opening up new strategies of investigating the heighting. Global Navigation Satellite System (GNSS) for 3D positioning is undergoing rapid developments and GNSS heighting can be an alternative to terrestrial techniques of height measurements. This paper presents a research study on the use of GNSS heighting in the case of steep slopes and multipath issue. Short baseline solution strategies were performed by using Bernese Software v. 5.0. The analysis results are also compared to the results of techniques of the terrestrial levelling. The results show that GNSS can be used as an practical surveying method to the terrestrial levelling with comparable accuracies. Furthermore, one can save up to 1 hour using GNSS instead of geometric levelling over a steep slope of a 100 m. On the other hand, as usual multipath is the primary error source decreasing the efficiency of GNSS, and it has been studied experimentally in this paper.

Keywords: GNSS Levelling; Height; Steep Slope; Multipath; Accuracy.

RESUMO

Na variedade de aplicações geodésicas, um dos principais focos tem sido em épocas recentes a determinação de altitudes acuradas de estações terrestres. Em especial, a possibilidade de aquisição de informação 3D com o posicionamento por ponto preciso, com alta precisão, está abrindo novas estratégias de investigação do posicionamento altimétrico com o *Global Navigation Satellite System* (GNSS). Este trabalho apresenta um estudo sobre o uso de posicionamento altimétrico com GNSS no caso de encostas íngremes com controle de multicaminho. Estratégias de solução de linhas de bases curtas foram realizadas utilizando Software Bernese V. 5.0. Os resultados dos processamentos foram comparados com os resultados de técnicas de nivelamento convencional para a avaliação da acurácia. Os resultados mostram que o GNSS pode ser utilizado como um método prático de levantamento para o nivelamento convencional, com erros comparáveis ao nivelamento geométrico. Além disso, pode-se reduzir significativamente o tempo de levantamento usando GNSS em substituição ao nivelamento geométrico convencional conforme testes apresentados sobre uma encosta íngreme. Por outro lado, como o multicaminho é usualmente a maior fonte primária de erro com implicação na eficiência do GNSS no tipo reportado de levantamento, este foi um dos principais focos de investigação nos experimentos.

Palavras-chave: Nivelamento GNSS; Altitudes; Encostas íngremes; Multicaminho; Acurácia.

1. INTRODUCTION

In engineering geodesy, studies based on the determination of heights are so common, for instance monitoring crustal deformation and deformation analysis of engineering structures such as high buildings and dams. Levelling is the common term applied to any of the various processes by which elevations of points or differences in elevation are determined. It is a vital operation in producing data for engineering applications. Vertical deformation has been measured using terrestrial geodetic techniques of geometric levelling and trigonometric levelling over years. Nowadays the GNSS is commonly used for civilian navigation, positioning, surveying and scientific applications as well as determining heights.

Geometric levelling is the simplest and the most accurate method to determine elevation differences. Occasionally, however, it can not be applied, e.g., for the determination of the elevation of a tower, or else is not economical, e.g., when crossed mountain ranges. For such cases, trigonometric heighting is applied. It is also generally recommended when elevation differences of lower accuracy are required since it is more economical. However, earth curvature and bending off the sighting ray due to refraction have to be considered for longer distances (WOLF and GHILANI 2002). Trigonometric heighting is gaining importance recently, since the slope distance can be measured very accurately and faster with total station instruments. The accuracy of trigonometric heighting depends on whether the influences of the earth's curvature and refraction are in a favorable ratio to the accuracies of vertical angle and distance. The influence of the earth's curvature and refraction can be neglected over short ranges. The low speed and systematic errors of geometric levelling with its horizontal lines of sight are reason enough for trying to replace geometric levelling by a trigonometric method with measured slope distances and vertical angles (KAHMEN 1988; WOLF and GHILANI 2002).

GNSS measures heights related to an ellipsoid, i.e. World Geodetic System-1984 (WGS 84). In some cases ellipsoidal heights alone are sufficient for the type of survey being undertaken. Clearly, the limitation in GNSS heighting is in the quality of the GNSS solutions used to gain heights. Major error source to consider is multipath. Reflective surfaces can mean that some of the signal reaching the antenna does not travel on a direct path from the satellite. Atmospheric delay is another issue to be considered. For a short baseline one can reasonably assume that the radio signals measured by both receivers pass through the same part of the atmosphere. However, as the baseline length increases that assumption begins to break down and atmospheric effects need more consideration. The other major source of error for GNSS heighting involves the antenna. The first and most clear problem is that the height of the antenna above the survey mark must be correctly measured (RIZOS 1997; HIGGINS 2000; FEATHERSTONE et al. 1998).

Some engineering applications strictly necessitate the use of orthometric heights (height above sea level or geoid) since physical definition of the height is important, such as in irrigation works. GNSS levelling could be a fast alternative in this case, however the distance between the ellipsoid and the geoid, called the geoid height or geoid undulation must be known accurately (HEISKANEN and MORIZ 1967). Depending on the accuracy requirements or the engineering application we use, GNSS levelling, especially a feasible method over long baselines, is preferable to the geometric levelling. One exception could be the use of geometric levelling for establishing higher order vertical control networks. In that sense, the accuracy of geoidal heights might not satisfy the standards/specifications of higher order vertical control accuracies.

Previously many studies, regarding the use of GNSS for height determination on various applications were presented and comparisons with terrestrial techniques were provided. First of all Featherstone et al. 1998 reviewed the accuracies obtained in GNSS heighting. There are studies that test the accuracy of GNSS determined orthometric heights using rapid static GNSS positioning (WU and LIN 1996; WU and YIU 2001). One of the versatile applications of GNSS positioning is the Real Time Kinematic (RTK) surveying, and orthometric height accuracies using the RTK positioning are studied by Featherstone and Stewart (2001). Fotopoulos et al. (2003) apply a network approach in determining orthometric heights using the GNSS and show how some systematic effects and datum dependent inconsistencies could be taken into account using a corrector surface. In rugged areas, orthometric heights could be derived from trigonometric heights, and this method could be used testing the accuracy of GNSS derived orthometric heights (HWANG and HWANG 2002).

A similar study was performed to derive geoidal heights from precise trigonometric heights across the city of Istanbul/Turkey which extends over a pretty rugged area (SOYCAN 2006).

For the aim of this study, applying terrestrial levelling techniques over slopes and rugged areas are somehow cumbersome since the surveying takes time and the accuracy is degraded. This study shows results from an experiment especially designed to show the performance of GNSS height determination over slopes in the case of multipath error. To show the applicability of the method results are compared with trigonometric levelling and geometric levelling. Furthermore, the results of several test trials concerning various data sampling and GNSS analysis strategies are presented. Thorough investigation is undertaken on multipath that is effective on one of the points used in the scope of this experiment since it is the primary error source on results.

2. DESCRIPTION OF THE EXPERIMENTS

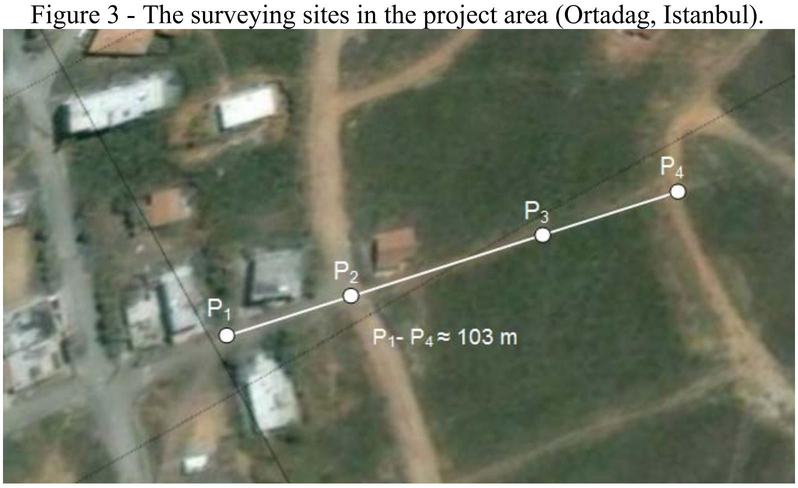
The experiments were performed in the Samandira region, Istanbul, Turkey. The geographic location of the studying area can be seen from Figure 1. In order to provide information on the a topographic model of the region, the ground elevation data (a digital terrain model) is presented in Figure 2. The digital terrain model (DTM) is based on the satellite images. The obtained data from the GIS Department of Istanbul Metropolitan Municipality, has been used for creating the maps and DTM of Istanbul. It gives the information on steep slopes at the studying area.

Figure 1 - Geographic location of the studying area.

Figure 2 - Digital Terrain Model for project area.

In order to test the performance the GNSS levelling versus the terrestrial techniques, we installed 4 GNSS sites, i.e. P_1, P_2, P_3 and P_4, along the selected profile over steep slope, see Figure 3. So the height differences between the sites could be derived from the **GNSS data processing**. Note that although the GNSS data were recorded for a different purpose during the first author's work in 2005, it was not used and published anywhere before.

Figure 3 - The surveying sites in the project area (Ortadag, Istanbul).



The GNSS measurements were collected using static surveying with two sets of Ashtech Z Max type receivers. As a reference station the site P_4 was chosen since it was the highest point of the project area thus having clear visibility of the sky, see Figures 3 and 4. The site P_1 was situated nearby a concrete building at about 6 m high and at a distance of 10 m (Fig. 4). P_2 and P_3 were located in fairly the free of multipath area. The distance between the starting site P_1 and the reference site P_4 was about 103 m. GNSS data was recorded in three sessions on the December 11th, 2005 (DOY: 345). Two Ashtech Z Max receivers were set up over the sites in the order of P_3 and P_4, P_2 and P_4, and P_1 and P_4, respectively (Figure 4). Surveying period for each session was approximately 2 hours. Note that the observation duration was 6 hours for the reference site P_4. For all the sites, the recording interval and the cut-off angle were set to 30 second and 15^o respectively. The key parameters can be seen in Table 1. The approximate location of the sites, are read by the receiver during the experiment.

Table 1 - Key Parameters of the GNSS Experiment.

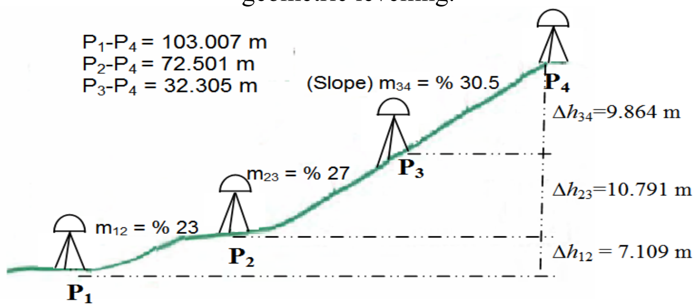
Date / GNSS Day	December 11th, 2005 / DOY 345
Session type	Static
Receiver type	Magellan Thales Z-Max NTRIP Receiver
Antenna type	**Z –Max Antenna**
Observables	C1, P2, L1, L2, S1, S2
Data interval	30 seconds
Elevation mask	5˚
Baseline lenght	~ 103.000 m
Height difference	~ 27.760 m
Approximate location of sites	P_1: 40° 57' 25" N 29° 13' 06" E P_2: 40° 57' 24" N 29° 13' 05" E P_3: 40° 57' 23" N 29° 13' 05" E P_4: 40° 57' 22" N 29° 13' 05" E
CODE final orbit	COD13530.EPH

Figure 4 - Four sites (P_1, P_2, P_3 and P_4) during the experiment.

Furthermore, terrestrial surveys, i.e. geometric levelling and trigonometric levelling, were also carried out to obtain independent results of the vertical position. The results of the terrestrial surveys were used to assess the accuracy of the GNSS solutions. To do it, the terrestrial measurements were performed using a total station, Leica TC 605 (with an angle measurement accuracy of ±5” and a distance measurement accuracy of 3mm + 3ppm) and a Topcon DL 102 digital level (standard deviation of which is 1 mm for double run levelling over 1 km). A barcode rod was used to determine the height differences. To eliminate common errors during the geometric levelling, a digital code level was used. The expected strong influence of the refraction was partly averaged out using automatic multiple readings by the instrument.

In Figure 5, elevation differences derived from geometric levelling are presented. Inclinations as percentage values are also given between the survey marks. Inclination values indicate how steep the slope over which the geodetic surveys performed is.

Figure 5 - The surveying profile of the sites and the height differences derived from geometric levelling.



The elevation differences obtained from the so-called terrestrial techniques, geometric levelling and trigonometric levelling are given in the first two columns of Table 2. The results of the terrestrial levelling were used as the reference values for assessing GNSS derived height differences along the following sections.

Table 2 - Height Differences from Terrestrial Levelling

Baseline	$\Delta h_{Geo.\ Levelling}$ (m)	$\Delta h_{Trig.\ Levelling}$ (m)
P_1-P_4	27.764	27.771
P_2-P_4	20.655	20.650
P_3-P_4	9.864	9.863

3. DATA PROCESSING

We installed one reference site (with 6 hourly data at the site P_4), and three rover sites (with 2 hourly sessions at the sites P_1, P_2, P_3) as mentioned above. The reference site was at the top of studying area, and away from the environmental effects in point of GPS levelling.

The coordinate solutions were computed by processing using Bernese GPS software version 5.0 (DACH et al., 2007). The data processing strategy was based on the double-difference approach. The positions of the local GPS sites were obtained by processing dataset including nearby International GNSS Service (IGS) permanent stations (ISTA, TUBI, SOFI, BUCU and POLV) and local sites. The Scripps Orbit and Permanent Array Center *(*SOPAC*)* was used to achieve GPS data of IGS stations, high precise satellite ephemerides and earth orientation in the Receiver INdependent Exchange Format (RINEX) with the sample rate of 30 seconds. As well known that CODE final orbit file (COD13530.EPH) related GPS week includes GNSS/GPS ephemeris/clock data in 7 daily files at 15-min intervals in SP3 format, including accuracy codes computed from a long-arc analysis, see Table 1.

For Bernese GPS software version 5.0, the general steps can be summarized as follows:

- Converting GPS data into Bernese format.
- Converting and interpolating CODE final orbits.
- Processing GPS code data to determine receiver clock error.
- Processing in carrier phase data to determine approximate coordinates of the sites.
- Selecting baselines and process baseline by baseline with coordinates fixed to determine carrier phase ambiguities. In this study, the baselines are about up to 550 kilometers and few tens of meters, for the long and short baselines, respectively.
- Fixed ambiguities and solving for coordinates.

In the first stage, *we performed* a successful ambiguity resolution *through* Bernese v.5.0 Software relative static long baseline processing strategies (DACH et al. 2007). The data set consists of IGS permanent stations and the site P_4 including 6 hourly static GPS data. We hold fixed ITRF-2005 coordinates of IGS stations in data processing. Phase ambiguities for L1 and L2 are solved by applying the quasi Ionosphere Free (QIF) strategy of the Bernese GPS software version 5.0 (DACH et al. 2007). The Niell (1996) dry mapping function is applied to map the a priori zenith delay (~ dry part), which is modelled using the Saastamoinen model (1973). The wet part of the zenith delay is estimated at a 2 hours interval within the network adjustment and it is mapped using the Niell wet mapping function. The International Terrestrial Reference Frame (ITRF) coordinates of the site P_4 were determined using data from the IGS permanent stations mentiond. In other words, the coordinates of the site P_4 were then introduced as known during a dual frequency processing of the whole sessions, including all the local stations.

Later, we processed local GPS *network* according to Bernese GPS software version 5.0 short baseline processing strategy (Dach et al. 2007). The data set now include GPS data of 6 hourly at the site P_4 and 2 hourly at the sites P_1, P_2, P_3. Note that ITRF-2005 coordinates of P_4 site coming from the processing of long baselines were held fixed. Ambiguities were solved using the sigma strategy (DACH et al., 2007). Local ionosphere models were created using the Bernese software. The troposphere was modelled using Saastamoinen apriori model mapped with Dry Niell mapping function. Zenith path delay paremeteres were estimated using Wet Niell mapping function at 2-hour intervals. The results of the ellipsoidal elevation differences between the local GPS sites from the static short baseline processing are given under the column of “Static Process (2 hr)” in Table 3.

Table 3 - Ellipsoidal Elevation Differences from Different Processing Strategies(m).

Baseline	Static Process (2 hr)	Standard Trop. (L1&L2)	Standard Trop. (L1)	Relative Trop. (L1)	Rapid Static (30 min)	Rapid Static (10 min)
P_1-P_4	27.782	27.776	27.774	27.764	27.775	27.773
P_2-P_4	20.662	20.653	20.656	20.652	20.653	20.656
P_3-P_4	9.871	9.862	9.862	9.878	9.867	9.872

We processed GPS data (i.e. three dataset of the two-hour sessions obtained using static surveying) using the other short baseline procedures of Bernese v.5.0 Software (DACH et al. 2007). The search ambiguity-resolution strategy, employing the Fast Ambiguity Resolution Apprroach - FARA (FREI and BEUTLER, 1990) and dual frequency GPS data, and the following Niell’s standard troposphere model and relative troposphere model were tested in the parameter estimation stage (NIELL 1996). We here tested following three different strategies as follows:

- L1&L2 signals + Niell’s troposphere model + Search ambiguity-resolution,
- L1 signal + Niell’s troposphere model + Search ambiguity-resolution,
- L1 signal + Relative troposphere model + Search ambiguity-resolution.

Note that, for tropospheric modelling, Bernese GPS software version 5.0 introduces the other standard models, i.e. Saastamoinen (SAASTAMOINEN, 1972), Hopfield (HOPFIELD, 1969) and so on, besides Niell’s model. If satellite elevation angle is greater than 10, Niell’s troposphere model gives similar solutions with the other standard models. The first two strategies we used to estimate the height differences was to employ merely a standard troposphere model (i.e. not estimating a relative troposphere bias between the stations). Here we assumed the troposphere is correlated over short distances and no need to estimate a relative tropospheric bias. This strategy was performed in two different stages (1) using only L1 and (2) using both L1 and L2. For relative troposphere modelling, tropospheric parameters were estimated in step of data processing (DACH et al. 2007). By this way, a relative tropospheric bias could be estimated depending on the assumption that the ionosphere is correlated over short distances. The ellipsoidal elevation differences between the sites are shown under the column of “Standard Trop. (L1&L2)”,

"Standard Trop. (L1)" and "Relative Trop. (L1)", respectively, in Table 3. Up to this point, through experimentation, we have determined 4 different value for each elevation differences from the processing strategies. These 4 values are varied to two centimeters, at the furthest.

Finally, the 2-hourly observations were subdivided into 30 and 10-minute sessions, and Bernese rapid static processing procedure was applied for 30 and 10-minute sessions, separately. Bernese GPS software version 5.0 manual suggests the successfully ambiguity resolution strategy for rapid static processing up to 10 km baselines. To do it, the Niell's standard troposphere model, the search ambiguity-resolution strategy and dual frequency GPS data were used (NIELL, 1996; FREI and BEUTLER, 1990; BEUTLER et al. 2001). The results for 30 and 10-minute sessions are given under the column of "Rapid Static (30 min)" and "Rapid Static (10 min)", respectively, in Table 3. Note that the elevation differences are computed by averaging of 4 and 12 separate solutions for 30 and 10-minute session durations, respectively. The carrier phase ambiguities are fixed successfully, the tropospheric delay is estimated from a standard model (Niell's model), and the final IGS precise ephemeris is used. The results agreed within 0.8 cm.

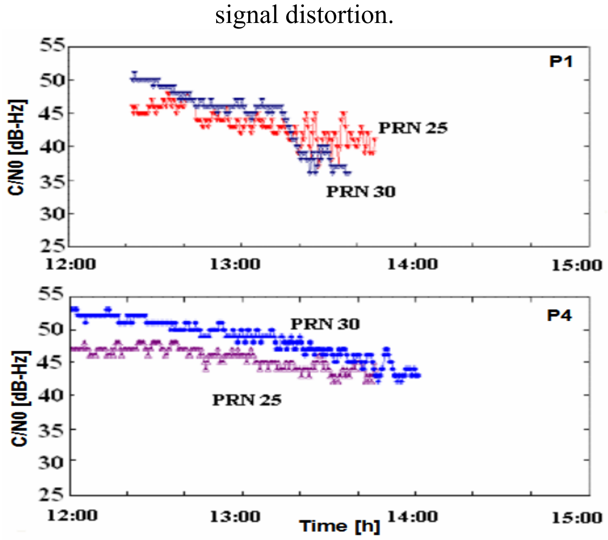
The elevation differences calculated from the GNSS measurements were compared with the height differences obtained from the terrestrial techniques. Note that, the contribution of geoidal height differences was neglected since the distance between baseline points is very short (e.g., $P_1P_4 \cong 103$ m) however one need to take this into consideration over longer baselines (Featherstone et al. 1998).

In Table 3, the maximum difference in acceptable scale between different processing strategies shows that the accuracy provided by the terrestrial techniques can also be obtained using the GPS. In fact, performing the terrestrial techniques over steep slopes is troublesome and time consuming since sight distances are limited due to rapid changes in elevation. In addition, in geometric levelling, the solutions are less accurate if the care is not taken in adjusting minus and plus sights. On the other hand, GNSS provides some results even with sessions as short as 10 minutes, see Tables 2 and 3. In this study, applying different processing strategies did not prove to be significant. The largest difference can be observed over the baseline P_1-P_4 however we think this is due to multipath error rather than the variation induced from applying a wrong strategy.

4. EFFECT OF SIGNAL DISTORTION

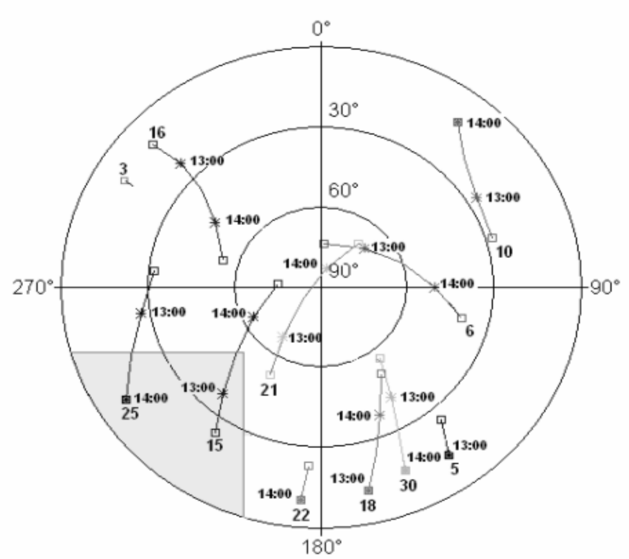
As seen from Table 3, the solutions of different processing strategies for the baseline P_1-P_4 fluctuate up to a few centimeters which is greater than the amounts listed for the other baselines and giving larger solution differences as compared to geometric levelling results. For studying the cause of this situation, two-hourly data set of three sessions, were evaluated using Ashtech Solution Software v.2.60 to analyze signal quality indicators in detail. In order to assess signal qualities, C/N_0 values are given in Figure 6. In this experiment, site P_4 is a favorably located reference site with almost no obstruction, while site P_1 is affected by an obstacle (i.e. the nearby building southwest of the GNSS antenna) which appears to degrade the quality of observations. The C/N_0 values obtained at P_4 change slowly with elevation only. The values at site P_1 (especially the C/N_0 value for PRN30) depart significantly from those at P_4 and vary independently of the change in elevation. Note that the colors of the C/N_0 plots for the same PRN numbers are not identical in Figures 6a and 6b. On the other hand, both stations are equipped equally and situated one hundred meter apart only, i.e. the satellite elevations are equal at P_1 and P_4 therefore the C/N_0 values should be equal. At the site P_1, the relatively low values indicate the effect of signal distortion, see Figsures 6 and 8. Especially, the standard deviation value of the height difference between P_1 and P_4 is significantly greater than the others. The concrete building for P_1 caused a significant bias in the phase measurements. Figure 7 shows the positions of GNSS satellites as observed by the antenna P_1. For instance, the antenna P1 was able to track the satellite signals of PRN 25 during 13:20-13:50 hours although propagation along the line-of-sight was not possible. The same assessment is also valid for PRN 15. These are typical cases of signal diffraction where a satellite signal is received, although the direct line-of-sight is obstructed. The C/N_0 values of PRN 25 indicate this diffraction effect for P_1, see Figure 6. During 13:20-13:50 hours the template suggests a C/N_0 value for PRN 25 around 50 dB-Hz for P_4 (see Figure 6b), however the actual C/N_0 measurement for P_1 is about 10 dB-Hz less. Beside the C/N_0 values, also the double-difference residuals (DD) display the diffraction effect (see Figure 8b).

Figure 6 - L1 C/N_0 of PRN 25 and PRN 30, measured simultaneously at P_1 and P_4 sites, at equal antenna/receiver combination. C/N_0 of observations at site P_1 indicate signal distortion.



PRN 30 is a medium to low elevation satellite, and its maximum elevation is about 60 degree (see Figure 7). The signal scatter is partially due to low elevation, but the distortion around 13:20 occurs when the satellite is close to its minimum elevation, see Figures 6 and 7. In addition the other satellites except PRN 15 and PRN 25 indicate the same pattern as PRN 30. Unfortunately, we did not give here all the values of C/N_0 for the other satellites due to the lack of space.

Figure 7 - Skyplot of site P_1 indicting the obstruction by the building over 12:25-14:30 UT.

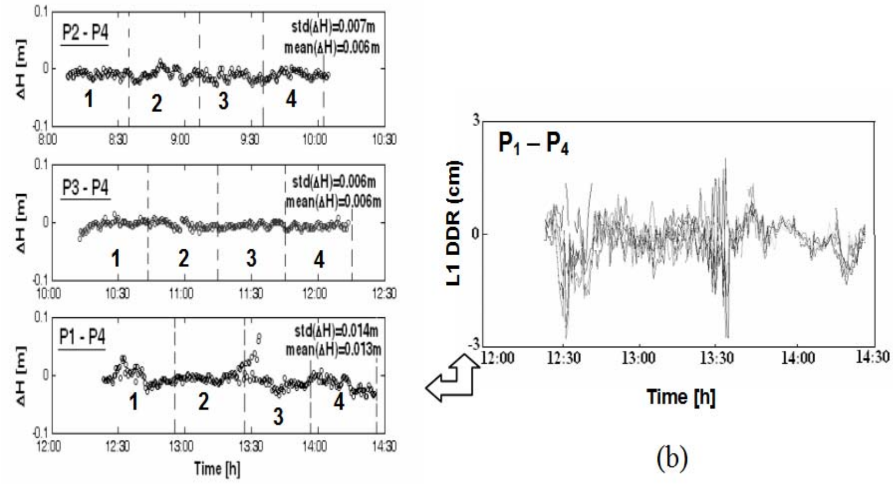


Next the diffraction effects on the coordinate results will be investigated. The results for the height component are displayed in Figure 8a. It shows epoch-to-epoch variation of this component minus the height differences from geometrical levelling. The corresponding mean values and standard deviations are also provided in the figure. As it can be seen Figure 8(a), for the line P_1-P_4, in the 1st, the 3rd and the 4th intervals (i.e. divided by the vertical dashed lines) the recorded average PDOP values varied between 2.0 and 5.2. For these intervals, the standard deviation and mean value of the height differences for P_1-P_4 are obtained above 1 cm, see Table 4. However, for the 2nd interval, the standard deviation and mean value of the height differences for P_1-P_4 are below 1 cm (Table 4) and the recorded average PDOP values varied between 1.4 and 2.0.

Table 4 - The Mean Values and Standard Deviations of the Height Differences (GNSS and Geometric Levelling) between the Sites.

Interval	Baseline		
	$P_1 - P_4$(m)	$P_2 - P_4$(m)	$P_3 - P_4$(m)
1	$std_{\Delta H}$=0.012 $mean_{\Delta H}$=0.011	$std_{\Delta H}$=0.005 $mean_{\Delta H}$=0.004	$std_{\Delta H}$=0.008 $mean_{\Delta H}$=0.006
2	$std_{\Delta H}$=0.005 $mean_{\Delta H}$=0.006	$std_{\Delta H}$=0.010 $mean_{\Delta H}$=0.007	$std_{\Delta H}$=0.007 $mean_{\Delta H}$=0.006
3	$std_{\Delta H}$=0.020 $mean_{\Delta H}$=0.018	$std_{\Delta H}$=0.008 $mean_{\Delta H}$=0.007	$std_{\Delta H}$=0.005 $mean_{\Delta H}$=0.006
4	$std_{\Delta H}$=0.010 $mean_{\Delta H}$=0.017	$std_{\Delta H}$=0.005 $mean_{\Delta H}$=0.004	$std_{\Delta H}$=0.004 $mean_{\Delta H}$=0.008

Figure 8 - The differences between the results of (epoch to epoch) GNSS and differential levelling as well as mean values and standard deviations of the differences for three baselines in (a) and L1 DDR values for P_1 - P_4 in (b).



(a) (b)

Furthermore, the L1 DDR values for baseline P_1- P_4 are shown in Figure 8(b). Between 12:25 UT and 12:50 UT, the maximum phase residual for the baseline P_1-P_4 is about 30 mm which is approximately 1/6 of an L_1 cycle. One should also note that the mean values and the standard deviations are below 1 cm for the height differences from P_2-P_4 and P_3-P_4, see Figure 8(a). Moreover, the L1 DDR values for P_2-P_4 and P_3-P_4 are significantly less scattered according to the ones for P_1- P_4. But, we can not show the residuals for all the sessions due to the lack of space.

From Table 4, it is clear that 10-min rapid static solutions corresponding to the period between 12:30 and 13:30 appear as spoiled for the baseline P_1-P_4, see the 3rd interval in Table 4. The few sessions for which this accuracy could not be obtained, corresponding to the sessions of bad satellite geometry and multipath effect on the site P_1 where the ambiguities could not be fixed. Thankfully, the rapid static solutions of 30-min do not expose such disturbances occurring at the same periods. The solutions for the baselines P_2-P_4 and P_3-P_4 do not include this kind of contamination since they are significantly far from the multipath environment on the site P_1.

5. A NOTE ON TIME SAVINGS

In this study, forward and return runs for the geometric levelling were completed in 1.5 hours. The height differences between the profile points were surveyed for about one hour using trigonometric levelling. Reciprocal zenith angle observations were conducted during the survey. As seen the results from Table 4, the short baseline static GNSS results obtained from 30 min sessions are as good as the ones obtained from the terrestrial surveying techniques. This study also shows that GNSS is as efficient as the terrestrial surveying methods in determining height differences over steep slopes from the point of view of time and economy.

6. CONCLUSIONS

This paper discusses two kinds of heighting techniques that are used to survey slopes: GNSS and terrestrial levelling techniques. The basic idea for an effective heighting over short ranges is to determine the elevation difference by using geometric or trigonometric levelling. However, surveying with these techniques over steep slopes might result in degraded accuracy due to some error sources, and also especially geometric levelling is time consuming. Moreover, some gross errors might occur. Therefore resurveying of the site is unavoidable.

The GNSS technology is widely used for many kinds of geodetic and engineering surveys. In this study, the use of GNSS heighting instead of terrestrial techniques over slopes was shown. Both (space and terrestrial) techniques were performed over the local test area, and GNSS derived height differences were determined. The results indicate that the differences between GNSS and terrestrial methods are obtained at about sub-centimeter for the experiments. This leads to a good heighting by the GNSS, and is less time consuming over steep slope. For the GNSS technique, there exists a specific disturbing effect, i.e. multipath, and it can corrupt the results of heights. Therefore the location of the local network is important, and one must avoid establishing the survey marks near the multipath areas. This method explained here could only be used over short distances, i.e. in our case it was only a 100 m. For longer distances, one has to include the effect of geoid-ellipsoid separation. For the future work, we are going to study the issue of the real-time kinematic GPS navigation in this scope.

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