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Research Paper

Accuracy Assessment of Horizontal and Vertical Datum Transformations in Small-Areas GNSS Surveys in Egypt

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ABSTRACT:
Global Navigation Satellite Systems (GNSS) surveys in small spatial areas might require simplified datum transformation and height conversion methods. The site calibration is one of several approaches that enable GNSS users to work with local coordinate systems and orthometric heights in real-time. This paper investigates the accuracy of such a method in different spatial scenarios utilizing geodetic datasets in southern Egypt. Based on the available data and attained results, it has been found that accuracy of quite a few centimeters could be achieved. Also, it has been noticed that this method performs better than the conventional geodetic datum transformation and geoid-based height conversion in small areas of less than fifteen square kilometers. Accordingly, it is concluded that site calibration is an efficient straightforward method to be utilized in GNSS surveys for engineering activities in small spatial regions in Egypt.
KEYWORDS: *GNSS, Datum Transformation, Height Conversion, Site Calibration.*

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I. INTRODUCTION

With the widespread of GNSS in Egypt and worldwide, the issue of datum transformation becomes more significant. It is a matter of reality that GNSS-based coordinates are relative to the World Geodetic System of 1984 (WGS84) which, in many cases, is not the national datum of a country. Consequently, a datum conversion procedure is needed to employ GNSS data in surveying and mapping on a national scale. Geodetic datum transformation is the mathematical process of defining the 3D spatial relationship between two geodetic datums. Such a process has been carried out in several countries in the last couple of decades, such as Indonesia [1], Ethiopia [2] , Croatia [3], China [4], Saudi Arabia [5] and Russia [6].

The task of datum transformation has been frequently investigated in Egypt in the last few years. For example, [7] (2000) have investigated the utilization of the multiple regression approach as a precise alternative for the traditional similarity transformation between the WGS84 and Helmert 1906 national Egyptian datum. In addition, [8] (2007) have utilized the regression datum shift concept to mathematically define the relationship between WGS84 and the local datums. [9] (2007) proposed improving the datum transformation process by applying the remove-restore technique. Recently, [10] (2018) have proposed the utilization of the Artificial Neural Network (ANN) technique as a novel model for increasing the accuracy of datum transformation.

On another hand, GNSS-based heights are ellipsoidal heights related to WGS84 while surveying projects depend on the orthometric heights relative to the Mean Sea Level (MSL) datum as an approximation of the geoid. Similarly, a height conversion procedure is vital in GNSS civil engineering and surveying activities. The vertical difference between these two datums is the well-known geoidal undulation or geoidal height. Therefore, a geoid or geopotential model is required to perform such a height conversion process. Since the 1960s, many Global Geopotential Models (GGM) have been developed and distributed by the International Center for Global Earth Models (ICGEM). So far, there exist 177 GGM models with variable characteristics, that could be free downloaded [11]. Out of the available GGM models, the Earth Gravitational Model 2008 (EGM2008) constitute a well-known model that has been utilized in height conversions worldwide [12].

The development of a national geoid model for GNSS height transformation in Egypt has been a major task for the geodetic community in the last few decades. Since the development of the pioneer national-scale geoid model by [13], several geoid models have been produced. For instance, [14] have computed a hybrid geoid model based on heterogonous geodetic datasets, that produced a standard deviation of 0.17 meter. Lately,

[15] have investigated the optimum combination of GGMs and global digital elevation models and developed a national geoid with an estimated accuracy of 0.13 meter.

The aforementioned traditional datum transformations, for both horizontal and vertical scenarios, are usually devoted to national-scale activities or large-areas GNSS projects. On the other hand, such transformation parameters and geoid models may not be available for all GNSS surveyors in some countries. Thus, other unconventional approaches have been developed and integrated into GNSS processing software to facilitate datum transformations in small-regions surveying projects. This paper aims to investigate the reliability and accuracy of such simple coordinate-conversion models particularly for small-areas GNSS surveys in Egypt.

II. GEODETIC DATUM TRANSFORMATIONS

3D Similarity or Helmert datum transformations contain, among others, the Bursa-Wolf and the Molodensky-Badekas models. The essential mathematical model can be stated, in a matrix representation, as (e.g. [16]):

$$X_T = c + sRX \tag{1}$$

$$X_T = \begin{bmatrix} X_2 \\ Y_2 \\ Z_2 \end{bmatrix}, \quad X = \begin{bmatrix} X_1 \\ Y_1 \\ Z_1 \end{bmatrix}, \quad c = \begin{bmatrix} dX \\ dY \\ dZ \end{bmatrix}, \quad R = \begin{pmatrix} 1 & R_3 & -R_2 \\ -R_3 & 1 & R_1 \\ R_2 & -R_1 & 1 \end{pmatrix} \tag{2}$$

Where,

XT and X are the coordinate vectors for the transformed and the original 3D coordinate systems respectively, c is the shift vector, s is the scale factor expressed in part-per-million (ppm) units, and R is the 3D rotation matrix containing the three small rotations R1, R2, and R3 about the X, Y, and Z-axis respectively. The Molodensky-Badekas model executes the rotations at an arbitrary point whose coordinates (Xo, Yo, Zo) are to be estimated too in the solution of the transformation parameters.

That model has seven unknown parameters in its general form. However, assuming small values of the three rotation parameters, it might be solved in four unknowns (three-shift parameters and a scale factor) and in other cases; three unknowns (shift parameters) could be enough for simplicity. [7] Have computed a precise set of transformation parameters from WGS 84 to the Helmert 1906 datums using the Molodensky-Badekas modeland found that:

Table 1: Transformation Parameters between WGS84 and Helmert 1906 [7]

Parameter	Value	Standard Deviation	Unit
dX	125.457	±0.41	m
dY	-113.943	±0.41	m
dZ	10.880	±0.41	m
R_1	-1.434	±0.23	second
R_2	-1.073	±0.42	second
R_3	5.088	±0.43	second
s	-5.4606	±1.08	ppm
X_o	4810523.5586	NA	m
Y_o	2925116.9363	NA	m
Z_o	2962668.8097	NA	m

It is a matter of fact that the traditional similarity datum transformation models assume that the relationship between two geodetic coordinate systems is uniform and can be modelled by a selected number of parameters. However, in reality, the actual parameterization is not as simple as implied in those models. For example, the old geodetic datum, which has been built up over years, does not have a uniform accuracy. Also, there exist distortions in the old datums as new precise data are fitted into these geodetic frames. Hence, the development of more precise relationships between geodetic datums should be considered [7]. The multiple regression technique depends on modelling the 3D differences in latitude, longitude, and heights ($\Delta\phi$, and $\Delta\lambda$, and Δh) between the geodetic coordinates of two systems by three polynomials of sufficient terms to represent the differences over the network, to a given degree of accuracy. This unconventional datum transformation technique has been used to describe the relationships between the WGS84 and 83 local geodetic datums all over the world where the basic formula could be stated as [17]:

$$\Delta\varphi = A_o + A_1U + A_2V + A_3U^2 + A_4UV + A_5V^2 + ... + A_{99}U^9V^9 \quad (3)$$

where,

Ao : is a constant,

A_o, A_1, ... A_{nn}: are coefficients to be determined,

$U = k(\phi - \phi_m)$: is the normalized geodetic latitude of the computation point,

$V = k(\lambda - \lambda_m)$: is the normalized geodetic longitude of the computation point,

k: is a scale factor and degree-to-radian conversion,

ϕ and λ: are local geodetic latitude and longitude (in degrees) of the computation point,

ϕ_m and λ_m : are mid-latitude and mid-longitude values (in degrees) of the local geodetic datum area.

Similar equations could be written for both $\Delta\lambda$ and Δh by replacing $\Delta\phi$ in the left side of the previous formal.

For instance, [7] have developed a multiple regression model for Egypt with a precision equals ±0.03 m, as:

$$\Delta\phi'' = -320.474 + 30.6751\,\phi_{84} + 3.0402\,\lambda_{84} - 1.7380\,\phi^2_{84} + 0.0436\,\phi^3_{84} - 0.0004\,\phi^4_{84} - 0.1056\,\lambda^2_{84} + 0.0012\,\lambda^3_{84} \quad (4)$$

$$\Delta\lambda'' = 4357.7294 - 734.6377\,\lambda_{84} + 49.4639\,\lambda^2_{84} - 0.1705\,\phi_{84} - 1.6600\,\lambda^3_{84} + 0.0278\,\lambda^4_{84} + 0.0037\,\phi^2_{84} - 0.0002\,\lambda^5_{84} \quad (5)$$

where,

$\Delta\phi$" and $\Delta\lambda$" are the differences, expressed in arc seconds, in both latitude and longitude respectively between the WGS 84 and Helmert 1906 geodetic datums, and ϕ84 and λ84 are geodetic coordinates relative to the WGS84 datum.

For GNSS surveys carried out in small areas, other simple datum transformation processes have been proposed. GNSS software manufactures have developed such models and integrated them into their processing prog rams. An example of such a method exist in the Leica Go Office (LGO) of Leica Geosystems and is called a none-step transformation [18]. It produces a set of transformation parameters that enable transforming the WGS84-based Cartesian coordinates (X,Y,Z) directly to a local datum projected coordinates (E,N). Another example of those processes is called site calibration developed by Trimble Co. [19]. It consists of a series of mathematical transformations to establish the relation between WGS84-based GNSS data and local control positions expressed as local projected grid coordinates along with MSL-based elevations. Those transformation procedures contain (ibid):

- A three-parameter datum transformation to convert the WGS-84 3D geodetic coordinates to 3D geodetic coordinates relative to the ellipsoid of the local map grid.
- A map projection to convert the local 2D ellipsoid coordinates into local 2D map grid projected coordinates.
- A geoid model to WGS-84 heights to obtain approximate elevations above the sea level.
- A horizontal least-squares adjustment of the transformed grid 2D coordinates to best fit local control data. This adjustment deals with any local variations in the projection system that cannot be accommodated in the overall datum transformation.
- A height least-squares adjustment converting the heights above the elevations derived from the geoid to the local MSL-based control elevations.

The site calibration, and other similar datum transformation processes, could be considered as unconventional solutions for coordinate conversion just in small areas. It can be concluded from the previous steps that the least-squares adjustment, for both horizontal and vertical coordinates, are utilized to best fit the GNSS-based coordinates to the local system. Such a fit would be valid only inside the project area and could not be applied outside such a small region. However, the merit of this solution is to give GNSS users to work in real-time with a local coordinate system since such a process could be performed during the field campaign.

III. METHODOLOGY AND AVAILABLE DATA

The collected GNSS database comprises 22 stations distributed over an area of 22 x 1 kilometers in southern Egypt. Those stations have known geodetic coordinates in both WGS84 and Helmert 1906 datums. In addition, their orthometric heights have been available too. Based on their spatial distributions, five different scenarios have been applied in the processing stage as presented in Table 2 and depicted in Figure 1.For each case, the site calibration has been accomplished utilizing at least four points that lay on the area's boundaries. Next, the coordinates of checkpoints are computed utilizing the attained site calibration outputs. Similar steps will be performed for the vertical site calibration too. On another hand, the site calibration outputs are compared against the results of utilizing traditional geodetic transformation and regression-based transformation models (Equation 1-2 and 4-5). Similarly, the vertical site calibration results of each case are judged against the global EGM2008 and the SRI2021 recent national geoid models. Those data processing procedures are depicted in Figure 2.

Table 2: Cases of Data Processing

Case	Region (km)	Area (km2)	No. of Calibration Points	No. of Check Points
1	23 x 0.8	18.4	5	17
2	17 x 0.8	13.6	5	15
3	13 x 0.8	10.4	4	13
4	8 x 0.8	6.4	4	12
5	0.6 x 0.8	0.48	4	10

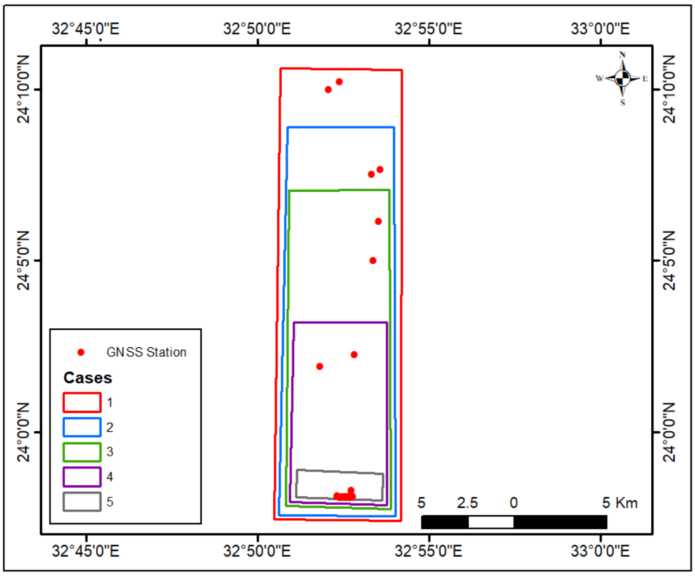

Figure 1: Data Processing Cases

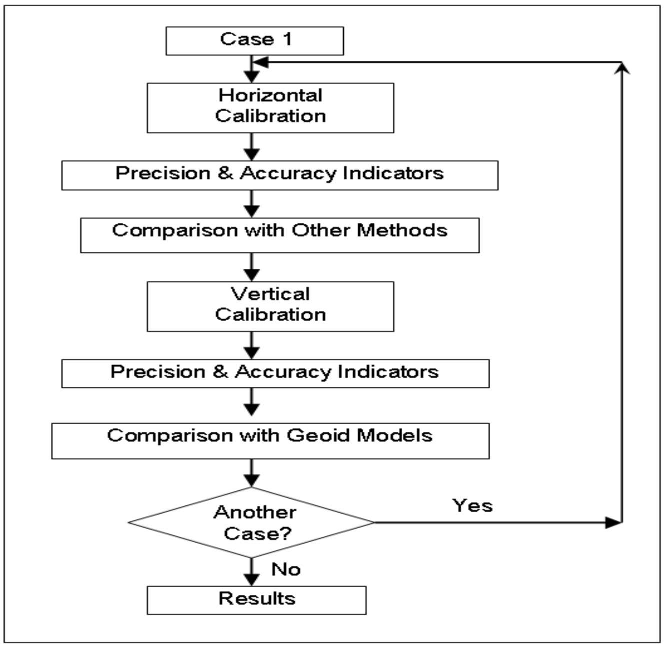

Figure 2: Processing Workflow

IV. RESULTS AND DISCUSSIONS

The accomplished results of site calibrations in terms of horizontal and vertical calibration are discussed in the next sub-sections. Additionally, the comparative results of site calibration with other transformation methods and geoid models are presented and analyzed.

4.1 Precision of Horizontal Transformations

The statistics of horizontal residuals of the site calibration process for the five different cases have been computed, tabulated in Table 3, and depicted in Figure 3. It can be noticed that the mean residuals vary between 0.020 m for case five and 0.045 m for case 1. The standard deviation of the horizontal residuals, as a precision indicator, ranges between ±0.005 m for the last case and ±0.021 m for the first one. Figure 3 shows that both average residual and standard deviation decrease with a small area of the project region as expected. That indicates that the smallest site calibration region, the better the precision of the anticipated output.

Table 3: Precision Statistics of Horizontal Calibration (m)

Case	1	2	3	4	5
Minimum Residual	0.023	0.023	0.001	0.002	0.026
Maximum Residuals	0.073	0.071	0.066	0.057	0.014
Average Residual	0.045	0.041	0.037	0.032	0.020
Standard Deviation	±0.021	±0.023	±0.030	±0.025	±0.005

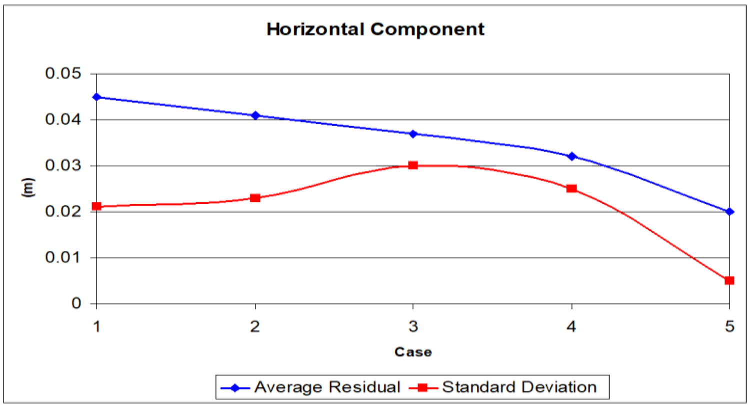

Figure 3: Precision Indicators in Horizontal Direction

4.2 Precision of Vertical Transformations

Similarly, the statistics of vertical residuals of site calibration for the five different cases have been obtained (Table 4 and Figure 4). It can be noticed that the mean residuals are almost zero for the five different cases. The standard deviation of the vertical residuals, as a precision indicator, ranges between ±0.001 m for the last case and ±0.021 m for the first one. Figure 4 shows that both average residual and standard deviation decrease with a small area of the project region as expected. That indicates that the smallest site calibration region, the better the precision of the anticipated output. However, the accuracy indicators are more reliable than the precision ones. The next sub-sections deal with estimating the accuracy indicators of the site calibration method.

Table 4: Precision Statistics of Vertical Calibration (m)

Case	1	2	3	4	5
Minimum Residual	-0.026	-0.011	-0.001	-0.001	-0.002
Maximum Residuals	0.027	0.012	0.001	0.001	0.001
Average Residual	0.000	0.000	0.000	0.000	0.000
Standard Deviation	±0.021	0.010	±0.001	±0.001	±0.001

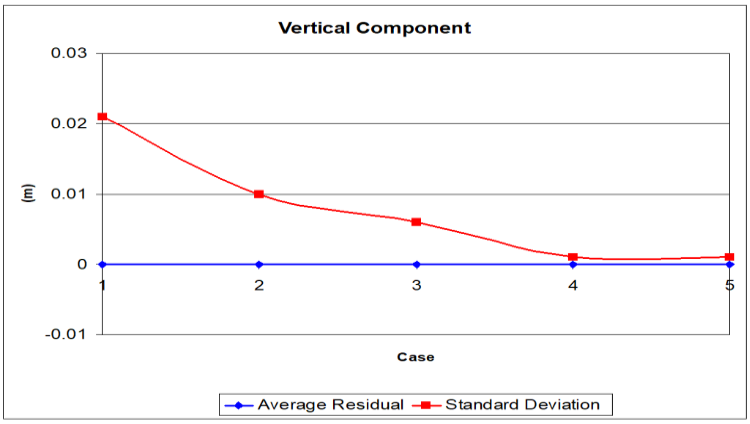

Figure 4: Precision Indicators in Vertical Direction

4.3 Accuracy of Horizontal Transformations

Over checkpoints, the coordinates resulted from the site calibration method have been compared against the corresponding known coordinates. The residuals and their standard deviations, in each case, provide a reliable accuracy indicator. For horizontal accuracy, Table 5 and Figure 5 and 6 present the accomplished findings. From this table, it can be noticed that the average residuals in the East direction range from - 0.183 m to 0.061 m and the standards deviations vary between ±0.348 m for case 1 and ±0.021 m for case 5. On the other hand, the average residuals in the North direction range from 0.153 m to 0.062 m and the standards deviations vary between ±0.236 m for case 1 and ±0.045 m for case 5. Thus, it can be concluded that accuracy of fewer than five centimeters could be attained in the smallest region (less than a square kilometer) as far as the available observations are considered.

Table 5: Accuracy Statistics of Horizontal Calibration (m)

Case	1	2	3	4	5
East Direction					
Minimum Residual	-0.773	-0.586	-0.113	-0.089	-0.001
Maximum Residuals	0.199	0.199	0.054	0.052	0.034
Average Residual	-0.183	-0.072	0.192	0.169	0.061
Standard Deviation	±0.348	±0.242	±0.110	±0.092	±0.021
North Direction					
Minimum Residual	-0.086	-0.079	-0.070	-0.063	-0.059
Maximum Residuals	0.739	0.753	0.032	0.027	0.008
Average Residual	0.153	0.149	0.147	0.117	0.062
Standard Deviation	±0.236	±0.243	±0.062	±0.055	±0.045

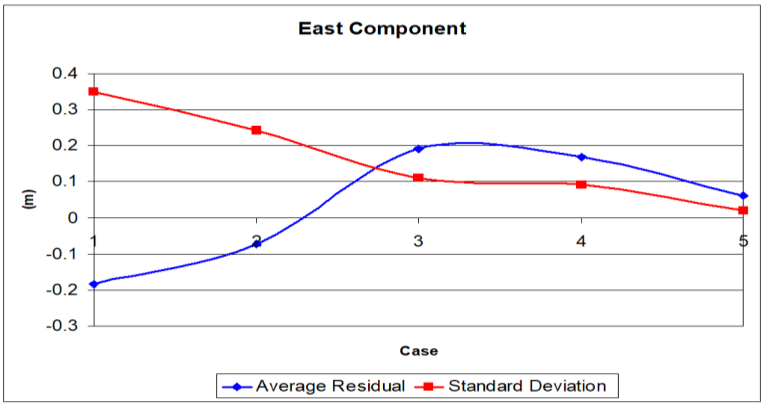

Figure 5: Accuracy Indicators in East Direction

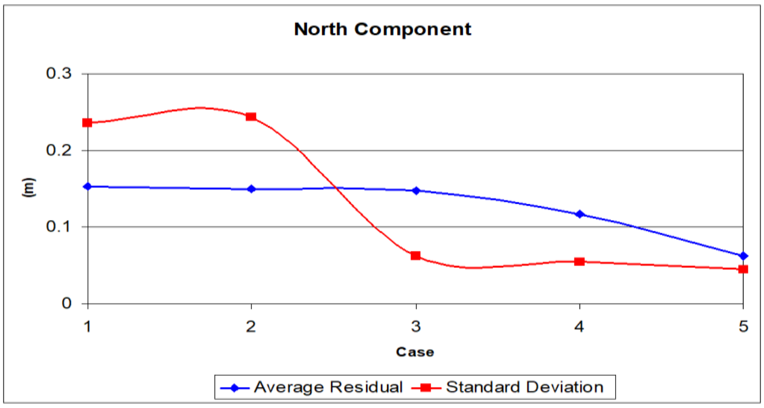

Figure 6: Accuracy Indicators in North Direction

4.4 Accuracy of Vertical Transformations

Similar to the horizontal coordinates, the heights obtained from the site calibration method have been compared to the corresponding known heights (Table 6 and Figure 7). So, it can be found that the average height residuals range from 0.150 m to 0.065 m, and the standards deviations vary between ±0.163 m for case 1 and ±0.019 m for case 5. In addition, the mean residual and standard deviation are decreasing as long as the area of the project region is decreasing. Consequently, it can be concluded that accuracy of fewer than five centimeters could be attained in cases 2 to 5 where the area of the study region varies from 14 to less than one square kilometer. More significant, it is noticed that accuracy of just two centimeters in height could be obtained from the site calibration method when the study area is less than a square kilometer.

Table 6: Accuracy Statistics of Vertical Calibration (m)

Case	1	2	3	4	5
Minimum Residual	-0.036	-0.070	-0.140	0.011	0.043
Maximum Residuals	0.417	0.092	0.128	0.104	0.104
Average Residual	0.150	0.042	0.061	0.054	0.065
Standard Deviation	±0.163	±0.043	±0.039	±0.032	±0.019

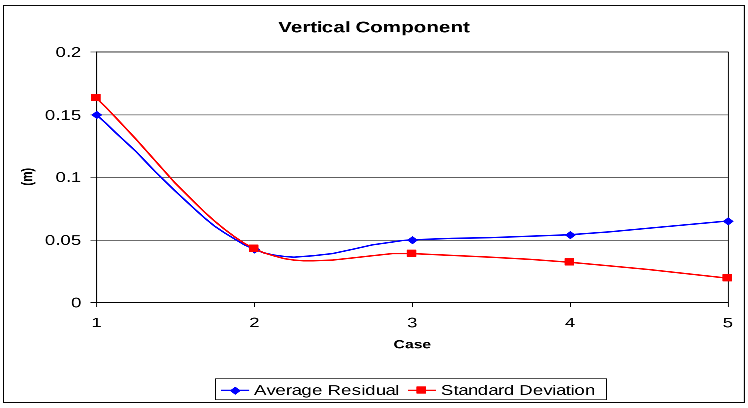

Figure 7: Accuracy Indicators of Vertical Component

Plotting the attained accuracy indicators of all cases against the area of the study region, Figure 8 concludes that the accuracy of the site calibration method is getting better as long as the project region is decreased. It is obvious, from this figure, that accuracy of almost ten centimeter could be expected for regions less than ten square kilometers. Five-centimeter accuracy, on the other hand, could be attained for study areas less than three square kilometers. Such findings conclude that the site calibration method should be utilized in GNSS surveying only for small-area projects.

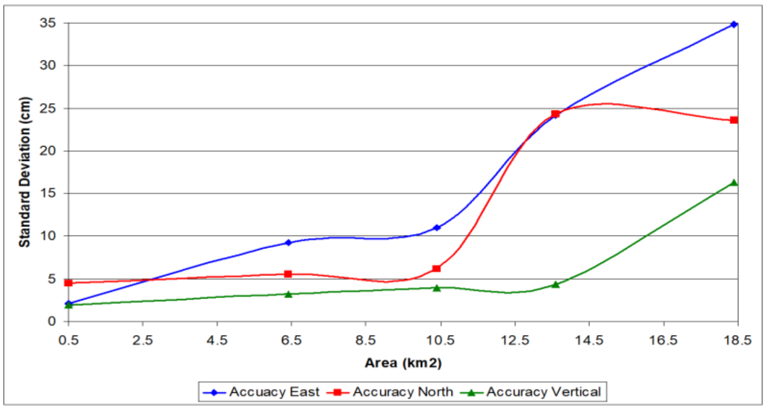

Figure 8: Accuracy Variations with Area

4.5 Comparisons to Other Methods

Having investigated the precision and accuracy of the site calibration method, the final analysis step is to compare its performance to traditional geodetic approaches in both horizontal and vertical components. First, the 3D conventional datum transformation using the Molodensky-Badekas model, with the parameters presented in Table 1, has been utilized over all available checkpoints. Next, the resultant geodetic coordinates have been projected on the Helmert 1906 local datum and, then, compared against their corresponding known 2D coordinates. Similarly, the multiple regression transformation (Equation 4-5) has been carried out. Table 7 and Figure 9 present the results of both datum transformation approaches over the entire study area. It can be realized that the first method produced an accuracy level, in East and North coordinates, varying between ±0.441 m and ±0.594 m while the second one produced accuracy standard deviations range from ±0.373 m and ±0.545 m. Comparing such findings with the corresponding results of the site calibration method (Table 5), it can be noticed that the results of site calibration in both projected coordinates are better than those of the traditional and multiple-regression methods in all investigated cases. This highlights the power of the site calibration for GNSS surveys in small areas. In another sense, both EGM2008 and SRI2021 geoid models have been used to interpolate geoidal undulations at known points, compute their geoid-based orthometric heights, and compare them to known heights. From Table 7 and Figure 9, it can be realized that the accuracy of both geoid models equals ±0.147 m and ±0.106 m respectively. Again, a comparison has been carried out between such results and the corresponding findings of the site calibration method (Table 6). Thus, it can be noticed that both geoid models perform better than the site calibration only in case 1 where the area of the project region equal 18.4 square kilometers. In all other study cases, the site calibration method is superior to the investigated global and local geoid models. Thus, it can be concluded that the site calibration method could be considered an optimum method for datum transformation and height conversion in small-areas GNSS surveys.

Table 7: Accuracy Statistics of Other Methods (m)

	Minimum Variations	Maximum Variations	Average Variations	Standard Deviation
3D Similarity Transformation: East	-1.155	-0.050	-0.677	±0.441
3D Similarity Transformation: North	-2.359	-1.185	-1.684	±0.594
Regression Transformation: East	2.217	3.241	2.659	±0.373
Regression Transformation: North	5.009	6.306	5.408	±0.545
EGM2008 Geoid	0.047	0.460	0.277	±0.147
SRI2021 Geoid	0.103	-0.258	-0.030	±0.106

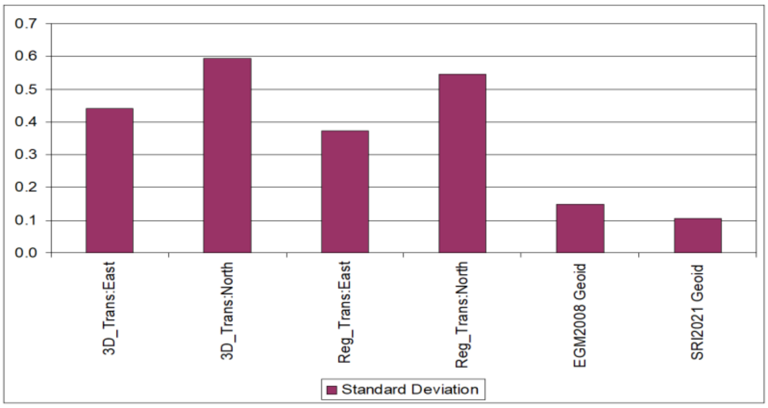

Figure 9: Accuracy Indicators of Other Methods

V. CONCLUSIONS

This paper investigates one of such untraditional datum and height conversions particularly for small-areas GNSS applications; the so-called site calibration. A geodetic dataset in southern Egypt, relative to both WGS83 and Helmert 1906 datums, has been utilized in several scenarios with variable lengths. It has been found that the precision of site calibration range from ±0.030 and ±0.005 in the horizontal direction and between ±0.021 and ±0.001 m in the vertical components. Over checkpoints, the accuracy of this method could be less than ±0.045m in both East and North coordinates, and ±0.019 m in heights particularly in areas less than a square kilometer.

A comparison has been carried out of those results to results of traditional geodetic datum transformation and multiple-regression approaches. It has been noticed that the site calibration method performs better than both approaches in all investigated scenarios. Another comparison has been performed between the heights resulted from site calibration and those obtained by utilizing global and national geoid models. For study regions less than fifteen square kilometers, the site calibration produces more accurate orthometric heights over checkpoints.

Based on the available data, it could be concluded that the site calibration method is optimum for converting WGS-84 geodetic coordinates to projected local coordinates and orthometric heights. Additionally, another merit of this method is to give GNSS users the capability to work in real-time with a local coordinate system since such a process could be performed during the field campaign.

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