

Discrepancies in Measurements of a Complex Street Intersection - Laser Scanner vs Accurate Total Station

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This study compares 36 point positions and 211 distance measurements completed with a modern, 12-second, laser scanner and a highly accurate, 1-second robotic total station. The latter serving as the benchmark instrument. The main objective of this quantitative comparison is to explore the accuracy and usability of a relatively large point-cloud model, as a virtual surveying tool for redesign/reconstruction purposes. This project involves the generation of a large, 3D, point-cloud model (240+ million points) of a complex city intersection, encompassing an approximate area of 300 ft × 750 ft and containing five converging elements: three streets and two railroads. It is an accident prone location requiring redesign. The resulting computer model has been geo-referenced in the Georgia East State Plane Coordinate System (SPCS) using control points with coordinates established by GPS (Global Positioning System) via a rapid, network-based, Real-Time-Kinematic (RTK) approach. The final point-cloud model was donated to the city to assist in the redesign of the intersection. A full analysis of the referred discrepancies is presented and recommendations on improving the overall current accuracies are provided.

Key Words: Construction Surveying, LiDAR, Laser Scanning, Total Station, Discrepancy

Introduction

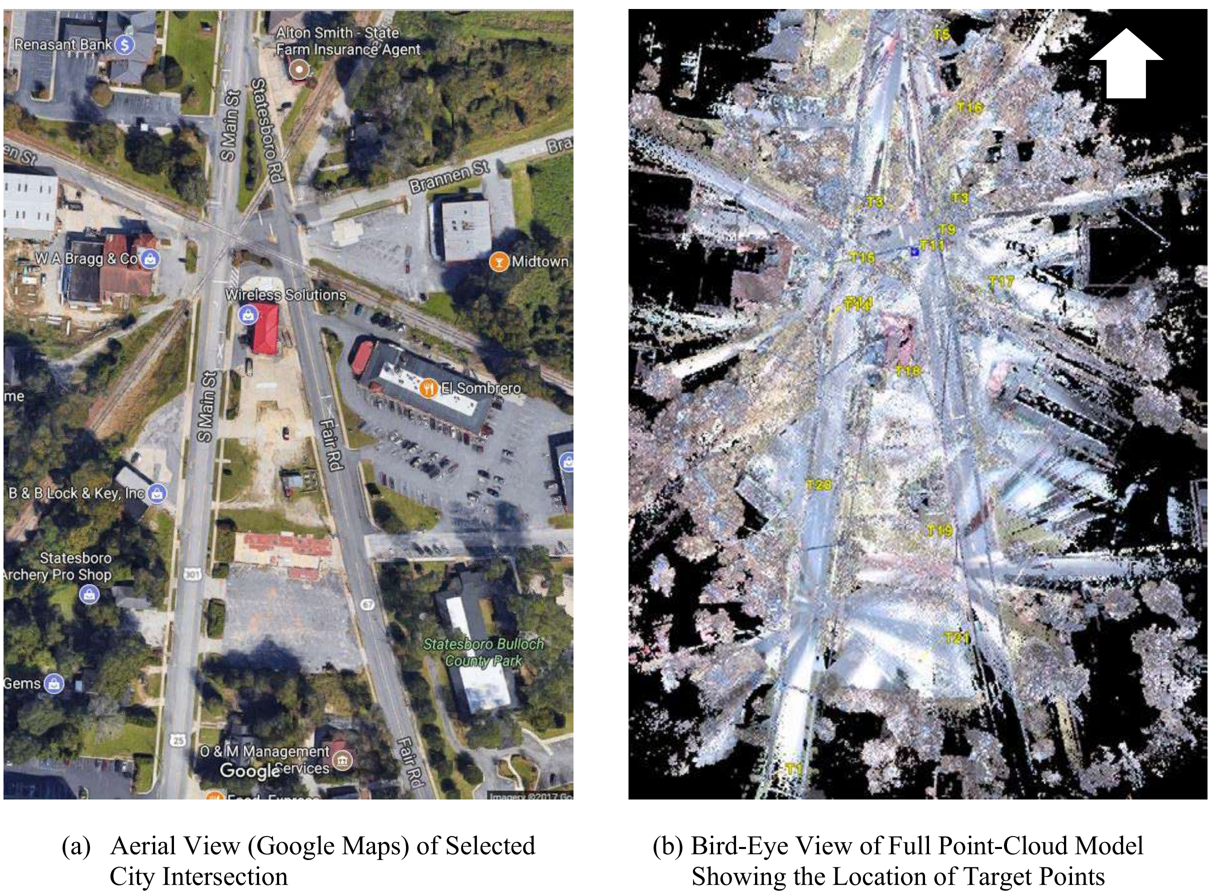
This work explores the ability of a modern, laser-based scanner to accurately model a relatively complex city intersection. For that purpose, various typical field measurements, assumed needed for the redesign, demolition and construction of a selected city intersection, are performed twice. One time with a modern laser scanner and a second time with a more accurate, laser-based, robotic total station, serving as the benchmark instrument. The authors investigate discrepancies in those measurements when performed by both devices. Today, laser scanners are sophisticated and powerful surveying instruments. Their technology shows continuous yearly improvements making them faster, lighter, more accurate, affordable and, consequently, more ubiquitous in design and construction firms across the Architecture Engineering and Construction (AEC) industry. In general, instrument manufacturers and sellers indicate the precision of these devices. However, these figures are not necessarily the same as the accuracies attained while performing regular field operations. Often, users do not have direct information on the actual accuracies they could attain when completing field measurements with laser scanners. This is confirmed by Huising and Gomes (1998) and by Simon et al. (2008). This work aims to expand existing information on this regard (Kersten et al., 2009). For that purpose, measurements obtained with a highly accurate, one-second, robotic total station are compared against the same ones obtained from a virtual point-cloud model generated with geospatial data collected by a modern, twelve-second, laser scanner.

A related previous study by Maldonado et al. (2015) compared discrepancies between measurements taken in a one-story building, with approximate plant size of 155 ft × 290 ft, using the same scanner as in the present work, but a different total station, a construction-grade, seven-second one. The latter instrument is less accurate than the one employed in the current study. That previous work was geo-referenced on an accurate, eight-sided traverse, enclosing the building. All total-station measurements were based on that reference frame and the resulting point-cloud was also geo-referenced in the same coordinate system. Results indicated that most of those measurements, ranging from 4 to 325 ft (≈ 1 to 100 m), had discrepancies within ±0.1 ft (±3 cm). This corresponded to an approximate relative discrepancy ranging from 1:30 for some of the short distances to 1:3,000 for the long ones.

The current work focuses on measurements performed not in a building, but in a relatively complex street intersection in the city of Statesboro, GA. The scanned area is approximately five times larger (about 300 ft × 750 ft) than the one considered in the previous study. Now, the reference framework is not a closed traverse, but several control points with GPS coordinates in the Georgia East SPCS, employing a survey-grade, GPS instrument and a rapid, network-based, RTK approach. The laser scanning instrument is the same in both studies, with an angular precision of ± 12 seconds. However, the total station employed in the current work is more accurate. It is a robotic, survey-grade instrument with angular precision of ± 1 second. This contrasts with the manual, construction-grade instrument, used in the previous study, with angular precision of ± 7 seconds.

Nine (9) undergraduate students from the Civil Engineering and Construction Management programs at Georgia Southern University participated in the present work. They collected and post-processed spatial data, acquired knowledge and experience, not only on the operation of advanced instruments (laser scanner and robotic total station) in the field, but also on the proprietary software package that accompanies those (2010). Students presented this project at the 2017 College of Engineering and Information Technology Undergraduate Research Symposium, in Georgia Southern, and were awarded the third place out of 90+ presentations.

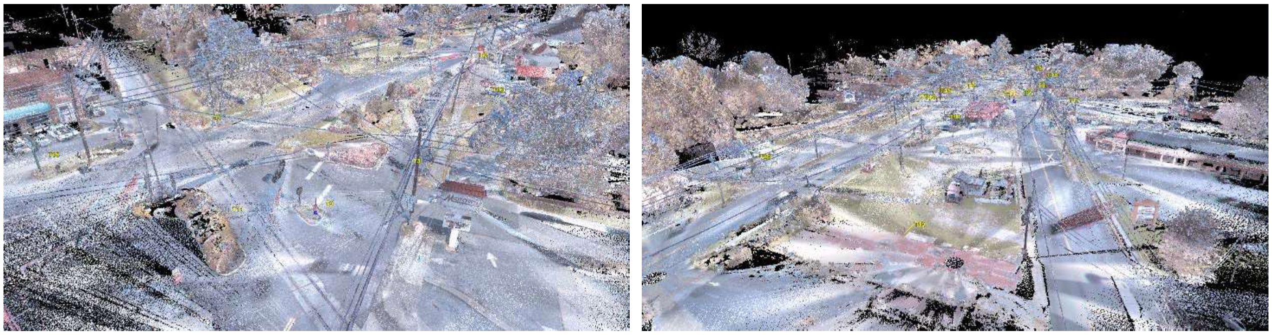
Aerial views of the selected intersection and its resulting final point-cloud model are presented in Figure 1 (a) and (b), respectively. Similarly, closer views of the final model are shown in Figure 2 (a) and (b).



(a) Aerial View (Google Maps) of Selected City Intersection

(b) Bird-Eye View of Full Point-Cloud Model Showing the Location of Target Points

Figure 1: Selected Intersection and its Associated Point-Cloud Model.



(a) View of the Intersection Toward Northwest (b) View from South Toward the Intersection

Figure 2: Final 3D Point-Cloud Model of the Selected City Intersection (with 3 converging streets and 2 railroads)

Objectives of this Study

The goal of this study is to explore the usability of a comparatively large 3D point-cloud model, generated with a modern laser scanner, for the geometric redesign and reconstruction of a city intersection. In particular, this work compares measurements obtained from within the virtual model generated by the scanner, against the same ones attained in the field with an accurate, survey-grade, one-second, robotic total station, serving as the benchmark instrument. The modeled field area is a relatively complex city street intersection, with three converging streets and, two railroads. Currently, this intersection is an accident prone location, requiring an improved geometric design and likely, reconstruction in a near future. Total stations are considered modern surveying instruments and, today, their manual or robotic models are the tools of choice for this type of measurements. Consequently, an accurate, robotic total station was selected as the benchmark instrument for this study. The referred discrepancy analysis involves determination of XYZ coordinates for various points within the intersection, and the calculation of numerous distances defined by them. The objectives associated to the above goal are as follows:

a) Obtain GPS coordinates of several points, near and within the area of interest. These points serve as control benchmarks with their coordinates in the Georgia East SPCS. These benchmarks are to be captured (scanned) by the laser scanning procedure.
b) Produce a 3D, full point-cloud model of the selected intersection employing a modern laser-based scanner.
c) Geo-reference the point-cloud model into the Georgia East SPCS by assigning the corresponding GPS coordinates to the scanned and captured benchmark points.
d) Use a highly accurate, laser-based, robotic total station to measure coordinates of selected points within the intersection area.
e) Complete discrepancy analysis by comparing coordinates and distances between points acquired by the scanner and the same ones acquired by the total station.
f) Make recommendations on the usability of the employed laser scanner to produce accurate models of similar size for design and construction purposes.

Employed Instruments – Comparison of their Capabilities

According to the manufacturer, the employed laser-based scanner is characterized by its long range, 300 m at 90% albedo (134 m at 18% albedo), ultra-fine scanning capabilities and its survey-grade accuracy. It captures spatial XYZ coordinates at a maximum rate of 50,000 points per second. The instrument presents an ample field of view with a full 360° horizontal coverage and a vertical-angle range of 270°. The standard deviation of its measuring errors (accuracies), within a 50 m range, are $\leq$ 6 mm and $\leq$ 4 mm for positions and distances, respectively. Its horizontal and vertical angular resolution, at one standard deviation, is 60 µrad (12 seconds). It presents dual axis compensators for precise automatic leveling of its vertical axis within 1-second resolution from zenith. This feature considerably reduces angular errors due to tilting of the vertical axis. This scanner also contains an integrated, auto-adjusting, high-resolution digital camera.

The selected robotic total-station instrument is capable of measuring with an angular accuracy of 1 second and with a reflectorless range of 1000 m. The standard deviation of its measuring errors (accuracies), for distances less than 500 m, is *2 mm + 2 ppm * Distance*. This accuracy decreases to *4 mm + 2 ppm * Distance* for distances larger than 500 m. This motorized instrument presents a robust centralized dual-axis compensator with setting accuracy of 0.5 seconds from zenith. As it was the case in the selected scanner, this compensator enhances the capability of this instrument to substantially minimize angular errors caused by tilting of the vertical axis. For ready comparison, Table 1 presents a summary of the main characteristics of the two instruments employed in this study.

Table 1: Comparison of Instrument Characteristics

Item	12-Second Laser Scanner	1-Second Robotic Total Station
Principle Type:	Pulse Based (Time of Flight)	Combined, Pulse and Phase-Shift Based
Range	**Reflectorless** **300 m** at 90% albedo. **134 m** at 18% albedo.	**Reflectorless: 1000 m**. (Using one standard prism, under light haze with visibility of 20 km, Range = 3,000 m)
Accuracy of single measurement	**<u>Reflectorless Mode</u>** Within **1-to-50-meter** Range: <u>Distance</u> (Std. Dev.) = **4 mm** <u>Position</u> (Std. Dev.) = **6 mm**	**<u>Distance, Reflectorless Mode</u>**: Std. Dev. = ± [**2 mm** + **2 ppm** × (Dist. **< 500 m**)] Std. Dev. = ± [4 mm + 2 ppm × (Dist. **> 500 m**)] **<u>Distance, Reflector Mode</u>**: Std. Dev. = ± [1 mm + 1.5 ppm × (Dist. **< 3000 m**)]
Angular Accuracies (Standard Deviation)	Horizontal Angle = **12 sec** Vertical Angle = **12 sec**	Horizontal Angle = **1 sec** Vertical Angle = **1 sec**
Inclination Sensor	Dual-Axis Compensator, with **1.5-sec** accuracy.	Centralized Dual-Axis Compensator, with **0.5-sec** accuracy.
Data collection Speed	Up to 50,000 points per second	Approximately, 1-3 points per minute

As observed in Table 1, when measuring distances within a 50 m (164 ft) range, the robotic total station is almost twice as much accurate than the laser scanner (2 mm vs 4 mm). Additionally, the robotic instrument has higher angular accuracy than the scanner (1 sec vs 12 sec). The latter has a direct impact on the magnitude of errors associated with position coordinates of acquired points. Position errors are affected not only by the distance from the instrument, but also by its angular accuracy. Thus, position errors increase with increasing distances and also increase when angular accuracies decrease. In this regard, the authors estimate that, when determining point positions, within a 50-meter range, the employed total station is close to 3 times more accurate than the scanner (2.01 mm vs 6 mm). The 2.01-mm, one-sigma error is estimated as the Pythagorean sum of 2.0001 mm (longitudinal error at 50-m distance) and 0.24 mm (error in transversal distance at 50m from device due to 1-sec angular error). This study presents comparisons of point coordinates determined using a polar approach, i.e. measuring distances and angles. Consequently, those coordinates and any associated distances are directly affected by angular accuracies.

Methodology

The following paragraphs describe the work flow completed to estimate discrepancies in position and distance measurements, when using the referred two different instruments.

Scanning and Registration Procedures

In this project, the scanned area is relatively large requiring 18 individual scans to fully cover it. To co-register the scans in the same coordinate system, they had to be stitched together. This process is referred to as registration and can be performed following one of at least three available approaches. In this work, *Target-to-Target Registration* was the procedure of choice. It requires to capture at least three common points, known as targets, from two neighboring scans to connect them together. Several types of targets were employed, i.e. 6-inch black & white, 6-inch spheres and twin targets. A set of instructions (protocol) was followed to complete each scan, including leveling the instrument, multiple target acquisitions and the scanning itself. Scanning was always performed at medium resolution (points separated by 10 cm at 100 m) and each had an approximate duration of 20 minutes. This time included the automatic acquisition of pictures by a built-in camera. Each scan contained targets with errors. The tolerance was set to 0.033 ft (i.e., 10 mm). Any target with errors higher than this limit were disabled allowing the model to become more accurate. This was possible by using redundant targets. Once, all scans were registered, a full 3D point-cloud model was created with an overall error of 10 mm and over 240 million points.

Acquiring Control Points and Geo-Referencing

Geo-referencing is the procedure where a final 3D model is aligned to a known geographical system of coordinates. In this case, the resulting point-cloud model was aligned to the Georgia East SPCS. For this purpose, GPS coordinates of seven ground points (nails) were acquired by personnel from the Georgia Southern Facilities, Services, Design & Construction Office. They employed a survey-grade GPS device and followed a rapid, network-based RTK approach. These points are referred to as control points. These same points were also acquired as target points (T1, T3, T5, T9, T11, T19 and T21) by the laser scanning procedure. Then, the already consolidated 18-scan model is adjusted to fit with minimum error the GPS coordinates of those seven control points. That is, the software is instructed to align the consolidated model into the coordinate system of the control points, Georgia East SPCS.

Finalizing the 3D Point-Cloud Model

Once the registration for geo-referencing was completed, each of the acquired 240+ million points had their XYZ coordinates in the Georgia East SPCS. Then, unnecessary/unwanted points were deleted. In this particular case, the 18 scans captured numerous passing cars, pedestrians and solar beams. Therefore, a set of procedures were followed to remove this traffic "noise" from the model, without affecting other points.

Distance Measurement and Discrepancy Analysis (3D Laser-Scanned Model vs. Total Station)

To determine the accuracy attained when measuring positions and distances within the final point-cloud model, a discrepancy analysis was conducted. It compared those positions and distances against the same ones measured in the field with an accurate robotic total station, the benchmark instrument. Initially, target location T9 (one of the control points), near the center of the intersection, was selected as a center point to measure 38 surrounding point positions (XYZ coordinates), and 38 distances, each from T9 to one of those 38 points. First, this was done in the point cloud where the location of T9 was easily identified by a tri-axial cross. The 38 surrounding points were selected among the available 240+ million ones so they were easily identified in the field and had uninterrupted lines of sight from T9. Examples of selected points are vertices of street signs, centers of letters in restaurant signs, vertices of buildings, etc. Then, in the field, the robotic instrument was stationed on the same T9 nail to collect coordinates of the same surrounding 38 points. The position of T9 is considered exact because its GPS coordinates coincide with the ones in the point-cloud model. Second, five additional center points were selected among the 38 surrounding points. Three of them far North of T9 (N1, N2 and N3) and two of them far South of T9 (S4 and S6). Then, 37 distances were measured from each center point to the remaining 37 surrounding points. This produced a total of 223 distances. They all were calculated using their coordinates in the point-cloud model and also by using their coordinates as obtained by the robotic total station. Two of the original 38 surrounding points showed inconsistent large discrepancies in their X or Y coordinates (0.44 and 0.45 ft). Therefore, they were considered erroneously acquired outliers and were not included in the calculation of the resulting statistics.

Results

After co-registering (stitching) all 18 individual scans, the resulting non-georeferenced point-cloud model presented an overall error of 0.033 ft (i.e., 0.4 inches) or 10 mm. However, the geo-referencing procedure increased this overall error to 0.101 ft (i.e., 1.2 inches) or 31 mm. This is due to the fact that each geo-referencing control point was acquired via a rapid RTK approach, stationing the GPS instrument for only about 15 seconds on each of them. This resulted in errors in their position coordinates, approximately ±1 inch in the horizontal components and about ±2 inches in the vertical component. Consequently, after geo-referencing, the inherent or minimum relative position error in this study is 0.101 ft or 31 mm.

Coordinate discrepancies were calculated for all selected 38 points by subtracting the coordinates acquired by the robotic total station from those captured by the scanning instrument. They are listed in Table 2 where two inconsistent outliers are observed, E8 and S5. They have component discrepancies of 0.45 ft and 0.44 ft, respectively. It was realized that those two points represented data erroneously collected in the field and, consequently, they were removed from the present study which was completed with the remaining 36 surrounding points. The ranges of these discrepancies (max and min values), their mean values, root mean square (RMS) values and standard deviations are summarized in Table 3. It can be observed that all three RMS values and their associated standard deviations range in magnitude from 0.05 ft to 0.09 ft (or from 0.6 inches to 1.1 inches). That is, about 15 mm to 27 mm each of them. This one-sigma error is consistent with the inherent error in this study.

Table 2: Coordinate Discrepancies for Selected 38 Points (including two outliers, E8 and S5).

Coordinates Discrepancies in Measured 38 Points (US-Survey Foot)									
#	Point Label	Northing Y (ft)	Easting X (ft)	Elevation Z (ft)	#	Point Label	Northing Y (ft)	Easting X (ft)	Elevation Z (ft)
1	*N1	-0.100	0.093	0.072	20	S3	-0.027	-0.078	-0.060
2	*N2	-0.037	-0.007	-0.036	21	*S4	0.065	-0.076	0.049
3	*N3	0.009	0.024	-0.017	22	S5	~~0.122~~	~~-0.443~~	~~0.069~~
4	N4	-0.123	0.007	-0.013	23	*S6	-0.067	-0.196	-0.013
5	N5	-0.111	-0.081	0.024	24	S7	-0.083	-0.082	-0.021
6	N6	0.004	0.031	-0.012	25	S8	0.139	-0.101	0.016
7	N7	0.123	-0.061	-0.136	26	W1	0.054	-0.046	-0.016
8	N8	0.018	0.031	-0.005	27	W2	-0.007	-0.059	-0.024
9	E1	-0.155	-0.041	0.001	28	W3	0.028	0.138	-0.096
10	E2	-0.009	-0.078	-0.066	29	W4	0.204	-0.095	0.055
11	E3	-0.133	-0.005	-0.089	30	W5	0.143	0.026	0.031
12	E4	-0.072	-0.056	0.028	31	W6	0.266	0.221	0.079
13	E5	-0.051	-0.026	0.015	32	W7	-0.012	-0.028	-0.012
14	E6	-0.021	-0.066	-0.007	33	W8	-0.001	0.023	-0.022
15	E7	-0.018	-0.022	-0.016	34	S9	-0.023	-0.049	-0.015
16	E8	~~-0.450~~	~~0.189~~	~~0.006~~	35	S10	0.007	-0.118	-0.041
17	E9	-0.005	-0.067	-0.121	36	N9	-0.022	0.040	-0.052
18	S1	0.039	-0.072	-0.025	37	N10	0.102	-0.012	-0.002
19	S2	-0.033	-0.058	0.003	38	S11	0.148	-0.120	-0.071

The measured coordinates of the selected center points (T9, N1, N2, N3, S4 and S6) are listed in Table 4. From each of these center points, a total of 35 distances (except 36 for T9) were calculated twice: (i) using coordinates obtained within the point-cloud model and (ii) by employing coordinates captured by the total-station instrument. This resulted in 211 different distances ranging from approximately 11 to 717 feet. Again, the corresponding discrepancies were calculated by subtracting the total-station distances from the scanned ones. Each major row of Table 4 shows results for a set of distances corresponding to a unique center point. Those rows are ordered by increased discrepancies in the location of their center points. This order shows some correlation with the column containing the RMS value of the associated discrepancies. All calculated discrepancies were plotted in Figure 3, where it can be observed that 63% of them (133) are in the ±0.1-foot range, with only 12 of them (5.7 %) exceeding the ±0.2-foot range. That is, the majority of the distances have a discrepancy within the inherent error of the model which is related to the geo-referencing control points.

Table 3: Range and Statistics Parameters of Coordinate Discrepancies for 36 Points (after discarding E8 and S5)

Item	Discrepancies in 36 Points		
	Northing Y (ft)	Easting X (ft)	Elevation Z (ft)
Maximum Value	0.266	0.221	0.079
Minimum Value	-0.155	-0.196	-0.136
Mean Value	0.007	-0.030	-0.017
RMS Value	0.093	0.081	0.051
Standard Deviation	0.093	0.075	0.048

Conclusions and Closing Remarks

In this study, the resulting point-cloud model was geo-referenced by employing GPS coordinates of seven control points. They corresponded to scanned targets T1, T3, T5, T9, T11, T19 and T21. These coordinates were acquired at the beginning of the study via a rapid, network-based, RTK scheme that triplicated the overall error of the virtual model, from 0.033 ft (≈ 0.4 in.) or 10 mm to 0.101 ft (≈ 1.2 in.) or 31 mm. In this work, this error is referred to as the inherent error of the model. The resulting spatial coordinates of numerous points in the selected intersection area, do not substantially differ if they were captured by either a laser-based, one-sec, survey-grade, robotic, total station or from the model produced by a less accurate, twelve-sec, laser scanner. After considering 36 points widely distributed within the modeled area (i.e., discarding 2 outliers), the standard deviations of the discrepancies in point positions almost coincide with their associated RMS values: RMS_{North}=0.09 ft, RMS_{East}=0.08 ft, and RMS_{Elev}=0.05 ft. That is, the standard deviations of those discrepancies range from 0.6 to 1.1 inches (or from 15 to 28 mm) in the considered intersection area. This is consistent with the inherent or minimum relative position error in this study, 0.101 ft or 31 mm.

Table 4: Analysis of Discrepancies in 211 Measured Distances from 6 Center Points, T9, N1, N2, N3, S4 and S6.

Selected Center Point	Employed Instrum. to acquire coords.	Coordinates of Center Point and their discrepancies			ANALYSIS of DISCREPANCIES in 211 MEASURED DISTANCES						
		Northing (ft)	Easting (ft)	Elev. (ft)	Discrepancy in Center Location, (ft)	# of Measured Distances	Max Discrep. (ft)	Min Discrep. (ft)	Mean Discrep. (ft)	RMS Discrep. (ft)	Std Dev Discrep. (ft)
T9	Scanner	887364.647	774166.884	219.084	0.000	36	0.185	-0.171	-0.024	0.083	0.079
	Total-Sta	887364.647	774166.884	219.084							
	Discrep.	0.000	0.000	0.000							
N3	Scanner	887579.637	774210.387	238.017	0.031	35	0.095	-0.299	-0.013	0.081	0.084
	Total-Sta	887579.628	774210.363	238.034							
	Discrep.	0.009	0.024	-0.017							
N2	Scanner	887531.928	774185.520	234.337	0.052	35	0.143	-0.338	-0.050	0.099	0.090
	Total-Sta	887531.965	774185.527	234.373							
	Discrep.	-0.037	-0.007	-0.036							
S4	Scanner	887030.972	774187.330	226.530	0.111	35	0.176	-0.210	-0.063	0.104	0.088
	Total-Sta	887030.907	774187.406	226.481							
	Discrep.	0.065	-0.076	0.049							
N1	Scanner	887634.145	773970.997	267.386	0.154	35	-0.046	-0.353	-0.148	0.159	0.073
	Total-Sta	887634.245	773970.904	267.314							
	Discrep.	-0.100	0.093	0.072							
S6	Scanner	887002.325	774307.979	228.228	0.208	35	0.292	-0.170	0.016	0.094	0.097
	Total-Sta	887002.392	774308.175	228.241							
	Discrep.	-0.067	-0.196	-0.013							

Regarding the discrepancies in distances, the coordinates of the referred 36 points were employed to calculate numerous distances between themselves and six points that served as centers (T9, N1, N2, N3, S4 and S6). A total of 211 distances, ranging from 11 feet and to 717 feet, were determined in this fashion, within the modeled intersection. Overall, most of them (63%) showed discrepancies within the ±0.10-foot range (±1.2 inches), i.e. within the inherent error of the point-cloud model incorporated by the GPS-based control points. 175 discrepancies, out of the 211 (83%), remained within the ±0.15-foot range (±1.8 inches) and 199 (94%) are within the ±0.20-foot range (±2.4 inches). Additionally, it is observed that the discrepancies of measured distances are not correlated to the magnitudes of those distances. The R-Squared value for these two variables is very low (R^2≈0.044). However, Figure 3 shows a tendency with negative slope as distances increase. Since total-station distances are subtracted from point-cloud-model distances, this could indicate that the resulting model tends to slightly underestimates distances as they increase in magnitude.

Finally, from a practical point of view, if the design/construction of an intersection, similar in size to the selected one, requires to work within one-inch accuracy, the procedure presented in this study is close to that requirement, but some distances may not be within that tolerance. Geo-referencing control points with low accuracy contributed to the observed discrepancies. Since the non-georeferenced model had a lower overall error (3 times smaller), it would have produced more accurate relative distances. If geo-referencing was necessary for design/construction purposes, the authors recommend to acquire highly accurate coordinates for the geo-referencing control points. This could reduce the magnitude of the inherent error 3 times with respect to the value observed in this study. In other words, if the coordinates of the geo-referencing control points were obtained with an accuracy of ±0.033-ft (±10 mm), it is expected that most virtual distances, extracted from the point-cloud model, will not defer in more than ±1 inch (±25 mm) from accurate field measurements completed with a survey-grade total station.

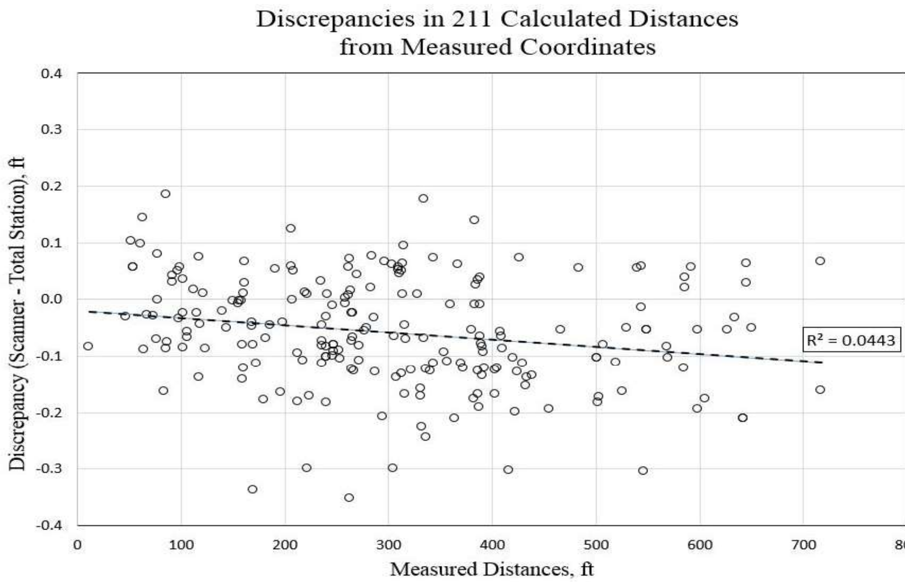


Figure 3: Discrepancies in Measured Distances from Chosen Center Points (T9, N1, N2, N3, S4 and S6)

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