PRECISION POTENTIAL OF UNDERWATER NETWORKS FOR ARCHAEOLOGICAL EXCAVATION THROUGH TRILATERATION AND PHOTOGRAMMETRY

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Commission II, WG II/9

KEY WORDS: underwater, photogrammetry, networks, precision, archaeology, SfM

ABSTRACT:

Given the rise and wide adoption of Structure from Motion (SfM) and Multi View Stereo (MVS) in underwater archaeology, this paper investigates the optimal option for surveying ground control point networks. Such networks are the essential framework for coregistration of photogrammetric 3D models acquired in different epochs, and consecutive archaeological related study and analysis. Above the water, on land, coordinates of ground control points are determined with geodetic methods and are considered often definitive. Other survey works are then derived from by using those coordinates as fixed (being ground control points coordinates considered of much higher precision). For this reason, equipment of proven precision is used with methods that not only compute the most correct values (according to the least squares principle) but also provide numerical measures of their precisions and reliability. Under the water, there are two options for surveying such control networks: trilateration and photogrammetry, with the former being the choice of the majority of archaeological expeditions so far. It has been adopted because of ease of implementation and under the assumption that it is more reliable and precise than photogrammetry.
This work aims at investigating the precision of network establishment by both methodologies by comparing them in a typical underwater archaeological site. Photogrammetric data were acquired and analysed, while the trilateration data were simulated under certain assumptions. Direct comparison of standard deviation values of both methodologies reveals a clear advantage of photogrammetry in the vertical (Z) axis and three times better results in horizontal precision.

1. INTRODUCTION

The main reason for establishing a common coordinate system, and realizing it by a control point network is the need for associating coordinates among re-visits on the site. The need for such networks in underwater archaeological excavations is apparent (Green et al., 2003, Nocerino et al., 2014). Archaeological excavation of a site usually stretches across many years and is revisited annually during the excavation period. In a similar way, in underwater archaeological excavations, establishing and measuring a network is a vital task when discovering and surveying an undisturbed archaeological site (Skarlatos et al., 2012).

A geodetic control network consists of stable, identifiable points with published coordinates derived from observations that tie the points together (United States Federal Geodetic Control Committee and Bossler, 1984). The main network of points is being densified with a secondary network of points closer to the working area. This is also typical in archaeological excavations, since the main network is being used to provide coordinates to a secondary network, in close proximity to the working area, i.e the excavation trench. The secondary network is being used in a daily basis during the excavation period, but control points of it are in danger of accidental movements or removal as the trench

progresses. Therefore, the accuracy and stability of underwater control points is of great importance. Indeed, the main network of control points must remain stable through the period of the excavation of the site, they should fully cover the area of interest, their positioning should be carefully selected to be far from the excavation area, where many tasks are performed and many divers work, yet no far away, which would increase measurements and work load during densification or daily surveying.

Sometimes this underwater network is georeferenced, but only when possible, such as using topographic methods with long poles in very shallow water or with floating buoys (Bass, 1966; Balletti et al., 2015; Diamanti et al, 2017). According to basic rules of error propagation, in order to preserve the precision of the local control network, georeferencing methods should guarantee at least the same precision of the control point network, otherwise will negatively influence the local coordinate system. When this is not possible then instead of geo-referenced measurements a local coordinate system is defined in relation to the site geometry. Georeferencing can only be performed properly in shallow water with the help of total stations or GNSS for depths up to 2-3 meters. Even if georeferencing is neglected, due to practical reasons or because it has no use, vertical reference is usually more important.

Depending on accuracy requirements and cost, vertical reference can be established more precisely using underwater laser levels for relative height differences measurements (Neyer et al., 2018).

2. NETWORKS

2.1. Geodetic Networks

Creating a network of control points is a threefold exercise; establish, measure and solve. Establishing a network comprises from the selection of locations, construction and marking of the control points. The next step is measuring the network, which is usually being done by terrestrial surveying using precision total stations or by satellite geodesy and GNSS receivers. The observations are either angles and distances in terrestrial surveying or time in satellite geodesy. The final step is the adjustment of all observations, to acquire the coordinates of the control points and their standard deviations from the variance-covariance matrix. The standard deviation of X, Y and Z are internal estimations of the precision of the calculated values, which are influenced by the geometry of the network, the quality and quantity of the observations.

2.2. Geodetic Networks in underwater environment

From the aforementioned three steps, only the latter one is trivial in underwater environment. Establishing a network is an extremely time-consuming task, with dubious results in terms of stability. Selecting locations of control depends on visibility, which is not always given. Distance among control points is deceiving in underwater environment, hence well distributed control points might not always be the case (Figure 1). Fixing the points for a long time cannot be guaranteed. Archaeological sites and finds are very fragile, therefore minimum intervention should be exercised. While a rocky sea bottom allows for stable fixation (Neyer et al., 2018), sandy sea bottom prevents any durable and reliable fixation. Indeed, as an excavation site is a working site with many divers passing and possibly moving heavy finds or using air-lifts, involuntary movement of control points cannot be excluded. Marking is by definition temporary, since sea salt and sea life will eventually decolorize any material.

Measuring such networks with terrestrial surveying or satellite geodesy, is not possible. There are two remaining techniques: trilateration using tape measurements or underwater photogrammetry using photographs taken by cameras enclosed in a waterproof housing (Nocerino et al, 2016). Tape measurements in the underwater environment pose significant difficulties and the precision is lower than in land (Holt 2003, Rule 1989, Atkinson et al. 1988). Key factors are sea currents, visibility and nitrogen narcosis. Sea currents bend the tape, requiring a lot of tension force to be exerted, which in a frictionless environment is very hard to apply. Unless full face masks are being used, visibility is the only mean of communication among divers. Hence, poor visibility restricts the maximum distance that can be measured by two divers. Nitrogen narcosis, which becomes evident after 25 m depth, affects reading and noting of distances. Nevertheless, trilateration is widely accepted as a tool for network measurement prior to photogrammetry (Casaban et al., 2014, Diamanti et al., 2017; Demesticha, 2011; Green and Gainsford, 2003). Horizontal coordinates are being calculated by trilateration, while vertical reference is attained in all cases by depth reading in wrist dive computers. This is because the coordinates on the Z axis are poorly estimated in an almost planar network of control points using only slant distances.

In a similar manner, underwater camera calibration is the main shortcoming in underwater photogrammetry (Menna at al. 2017). Under-the-water, the mathematical model based on collinearity equations and standard radial and decentring distortions used above the water, also known as single view point pinhole camera, may not hold anymore because of the refractive effects caused by multiple media involved (water-glass-air). In certain circumstances, using corrective optical elements (i.e. when using a dome port) the use of the pinhole camera model may be a sufficient approximation (Menna et al, 2018). In these cases, self-calibration, implemented in most Structure from Motion (SfM) techniques, does significantly simplify the process of measuring underwater provided that a control point of network is used to independently check the absence of systematic residual errors in 3D measurements by photogrammetry. This, along with Multi View Stereo (MVS), made the otherwise extravagant photogrammetry, a popular tool among archaeologists (Skarlatos et al., 2012, Demesticha et al., 2014, Casaban et al., 2014).

Given the rising popularity of photogrammetry in underwater archaeological applications, and the wide use of SfM and MVS as an accurate tool for 3D modelling, the need for proper control points was highlighted. This study focuses on the precision of the two prevailing methodologies for control point network measuring; tape measurements and photogrammetry.

Despite the adoption of underwater photogrammetry as a precise and accurate 3D modelling tool, there are several research archaeological expeditions using trilateration adjustment to assign coordinates to control points. The current situation in underwater networks has been described in Skarlatos et al. (2017), but the most detailed trilateration analysis so far has been presented by Neyer et al., (2018), where a coral area of 16 x 8 m and 3.8 m maximum height difference, was covered by 9 Ground Control Points (GCPs). Authors report standard deviations σX, σY and σZ of 5.8, 8.5 and 9.7 mm respectively, for a free network adjustment. The depth of the site is not reported, and maximum distances measured for trilateration were less than 10m, as interpreted by the published network figures. In the continuation of their work, the authors reported that after improving the distance and height differences measurements a final average standard errors of 1.3 mm in planimetry and 1.5 mm in height were achieved for a plot of 5mx5m at 10 m depth (Nocerino et al., 2019). While highly accurate geodetic networks are attainable underwater, they require massive time efforts, which are not practical at deeper depth or for archaeological expeditions.

This work focuses on the analysis of precision estimation provided by trilateration and photogrammetry in a typical underwater archaeological site. The geometry of the control network and the extension of the site differ significantly from what presented in Neyer et al., (2018). Other constraints include the impossibility to place GCPs in the middle of the archaeological site, depth and dive time limitations, hence limited in total resources for proper trilateration measurements. Therefore, the triangulation adjustment was simulated limiting the possible measured distances to 20m length maximum.

3. METHODOLOGY

3.1. Test case

In order to perform an error analysis over a typical archaeological shipwreck documentation network, an example

case of an approximate wreck area 14m by 21m, and random orientation was selected (Figure 1). The network consists of signalised 8 points surrounding the area of the wreck. The average depth of the wreck is 27m, with signalised point depths varying from 25.5 to 27.m depth. The maximum stretch among control points is 23.1m, among points #4 and #7. The minimum distance along the perimeter is 4.0m (points #3-#4) and maximum 13.0m (points #1-#7).

The signalised points are positioned in the perimeter of the exposed wreck, with an additional buffer zone to compensate for any possible finds buried under the sand. The buffer zone may vary but, typically, 1m should be considered the minimum. The network design cannot support points in the center of the area, since, most certainly, any attempt to fix points there will result in damaging or at least disturbing finds. For this particular case, sea bottom is sandy, almost flat with a stable inclination, and the wreck area is surrounded by posidonia oceanica seaweeds. For the control point marking, 2-inch PVC pipes were hammered 20-30 cm in the sand on average. The remaining height of the poles was 30-20 cm. A plastic tap with a retroreflective target, was screwed on the pipe, at the end of the pole (Figure 2).

Aim of the established network is to provide support for all excavation and monitoring purposes. The network of signalised points is further densified closer to the trench during the excavation period. Additional control points are then established on the vicinity of the trench, to support more detailed and close range photography, in order to record 3D details of the finds and better 3D modelling. All further densifications are carried out photogrammetrically, with dependency over the initial control network, whose accuracy is of utmost importance. This is a typical scenario for an underwater archaeological excavation, although the extension of the area (both in planimetry and depth), sea bottom morphology and depth might vary considerably.

For both surveying techniques, trilateration and photogrammetry, it is crucial to define the precision of the raw measurements. In photogrammetry, the precision that an image point is measured, is easier to define. Typically, for manually measured image points, less than a pixel, in the order of ⅓ to ¼ pixels (Kraus, 1997), ½ pixels (McGlone, 2004) are reported. As a rule of thumb, usually ½ or 1 pixel is adopted for such studies. In our case we selected 1 pixel as standard deviation of image measurements. This is the maximum that appears in literature, in order to compensate for calibration residuals and image degradation effects, typical in underwater photography.

The standard deviation of tape measurements is more difficult to define, as it is influenced by several factors, the main being the tension that is needed to avoid gravitational bending. In the underwater environment, the gravitational bending is less prominent because of the buoyancy. Sea currents may also significantly alter the straightness of the tape. In addition, divers cannot exert enough force to the tape, particularly in archaeological sites, where divers should avoid as much as possible touching the bottom as they might harm archaeological evidence or even move the control point pole itself. Therefore, even the slightest current creates a bending curve. As a countermeasure, long distance measurements should be avoided.

As the depth increases, divers are prone to nitrogen narcosis, which may turn an easy task into a complicated one. At depths of 40m, divers may experience difficulties in taking readings

from the tape and writing them down properly. Holt (2003) reports that even at 10m depth, 18.8% of measurements is rejected as outliers.
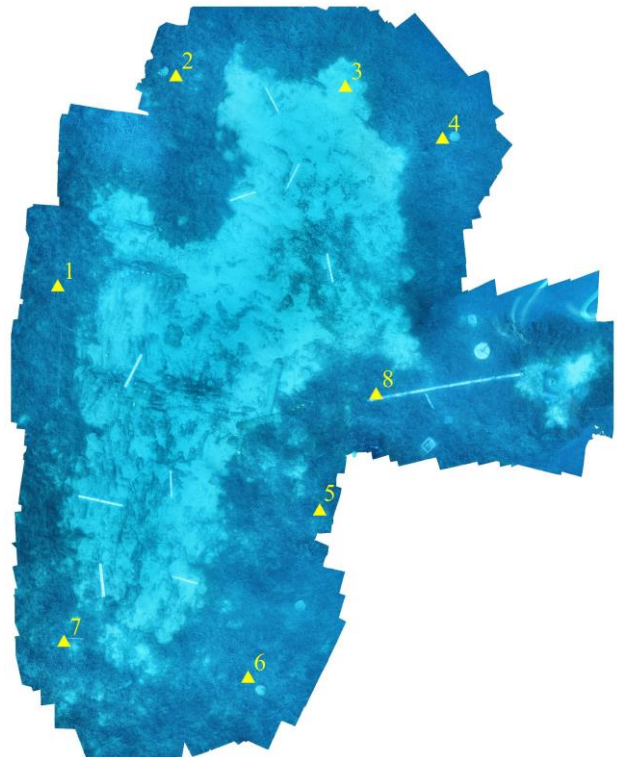

Figure 1. Overview orthophoto of the shipwreck site and overlaying control points.

Figure 2. Control point fixation and marking.

At the same study it is estimated that up to depths of 10m, the standard deviation of tape measurements of up to 20m distances, is 25mm, after outlier removal. Atkinson et al. (1988), studied a 14x3m wide area at 30m depth, using 15 control points and report that a realistic expectation of tape measurement precision is 0.05m. They also reported of high

outlier rate and necessity to re-measure certain distances. Large standard deviation was assigned to remaining dubious measurements. Rule (1989) reports that for 23 measurements taken at 12m depth, the average standard deviation is between 0.3% and 0.4% of the distance.

For the context of this simulation, the standard deviation cannot be fixed for all distances, since the bending error should be proportional to the distance measured. The depth of this particular example is also more that the cases reported in Holt (2003), Rule (1989) and Atkinson et al. (1988). Also, the distances measured were bigger that the ones reported, hence we adopted the worst case reported, i.e. 0.4% of the distance. This represents error of 0.02m in 5m distance or just 0.08m at 20m. It should be noted that this is based on 68.3% confidence level, meaning that larger error might appear for the remaining 31.7% of the observations.

4. APPLICATION

4.1. Trilateration

GaMa (Cepek, 2002) GNU project was used for free network adjustment of trilateration. Having had coordinates of the control points, all three dimensional Euclidean distances were calculated. The network was treated as three dimensional, meaning that observed distances were slant distances and the unknowns the X, Y and Z coordinates of the control points. Distances above 20m were removed from observation data set, as being impossible to be measured reliably at 27m depth (Figure 3). Adopting the aforementioned standard deviation of 0.4% of distance, a Gaussian noise was added to all distances. In addition, the adjustment was performed using weights based on the assumption that observed distance have 0.4% standard deviation.

Each distance among two given points was included twice in observation data set, with different noise, as it was considered that for reliability reasons it would have been measured twice underwater. It is typical in such networks to fix one point and one direction (minimal constraints). In this type of solution, the fixed point has zero standard deviation (as it remains known and fixed), while the point with the highest standard deviation is in the opposite side of the network, since it accumulates all network and measurement discrepancies. This is a very common approach employed in monitoring applications (Neyer et al., 2018). However, in this study, a free network with inner constraints was adopted. This solution provides optimal results in terms of inner coordinate accuracy, minimizing the mean variance of point coordinates (i.e., the cofactor matrix Qxx has minimal trace compared to all others adjustments with minimum datum).

As shown in (Figure 3) this also leads to a more balanced distribution of standard deviations and error ellipses. In this way, all points of the network are adjusted and assigned a standard deviation. In order for the results to be invariant to the artificial gaussian noise vector, the process was repeated ten times, with different random gaussian noise vectors and the included in Table 1.

The observations were 44 (22 distances among points, each one observed twice), and the unknowns 24, hence 20 Degrees Of Freedom (DOF). Since this was a free network adjustment 6 more degrees of freedom should be added, for a total of 26 DOF. The average σo of the adjustment was 0.653.

The standard deviations (internally estimated error of calculated coordinates) reveal that the point with maximum uncertainty is #7, with σXY 0.10 m. The average σXY error is 0.06 m and the average σZ error, is 0.64 m.
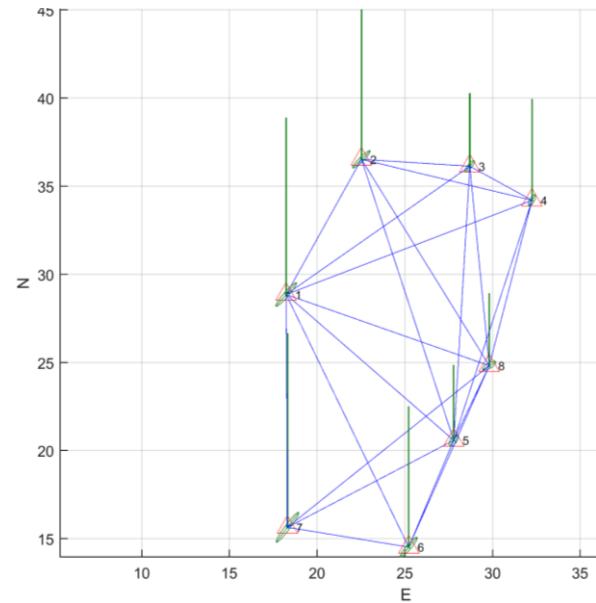

Figure 3. Typical results from one of the ten trilateration adjustments. The error eclipses are scaled ten times. The standard errors in Z (green vertical lines) are significantly larger than X and Y ones.

id	X	Y	Z	σX	σY	σZ	σXY
1	28.815	18.187	-26.348	0.0613	0.0539	0.9153	0.0816
2	36.528	22.531	-26.976	0.0451	0.0454	0.7755	0.0640
3	36.142	28.724	-27.488	0.0299	0.0247	0.3780	0.0388
4	34.211	32.277	-27.656	0.0301	0.0192	0.5239	0.0357
5	20.596	27.786	-26.179	0.0305	0.0239	0.3894	0.0387
6	14.456	25.173	-25.670	0.0511	0.0446	0.7288	0.0678
7	15.746	18.399	-25.757	0.0775	0.0591	1.0026	0.0975
8	24.849	29.855	-26.926	0.0239	0.0342	0.3753	0.0418
				0.0437	**0.0381**	**0.6361**	**0.0582**

Table 1. Averaged results from ten repetitions of trilateration with different random gaussian noise. All results in m.

4.2. Photogrammetry

MetaShape (previous PhotoScan) from Agisoft with DBAT[1](Börlin and Grussenmeyer, 2013) Matlab script, were used to process the photogrammetric measurements. Ten bars of up to 4.5m were used for scale, in an 8-parameter self calibration bundle adjustment with 295 photos. A Canon EOS 550D with a 20mm zoom lens was used for photo capture, which is a mid-range camera with an equivalent lens. Total area covered was 230 square meters with an average ground pixel size of 1mm. The automatic sparse tie point cloud was cleaned for blunders using gradual selection and manual selection for points away from main concentration. In total, 290K tie points remained, with 880K image projections. Overall, blunder rejection was basic and by no means thorough, as the final 1 pixel re-projection error suggest. Although the photos were

[1] https://github.com/niclasborlin/dbat/

taken vertical, the layout of the photos was unconventional to aerial practice, mainly due to dive time constraints and depth maintaining experience. In short, data acquisition was oriented towards archaeological practise and by no means focused on strict photogrammetric methodology. The average scale error was 0.001 m. The initial BA results from MetaShape were exported to DBAT for further processing. In its current implementation, DBAT does not allow for a free network with inner constraint solution. A minimally constrained BA process was then performed by fixing the 6 degree of freedom (DOF, i.e. the exterior orientation) of one camera plus one distance to another camera (baseline). The most central photo of the block was selected to fix the exterior orientation and the distance to the furthest camera was defined to fix the scale, data being selected from the MetaShape solution. A further image observation cleaning step was performed by removing all the 3D tie points triangulated with an angle smaller than 10 degrees and visible in only two image. Finally, the standard deviation of all signalised points was estimated in DBAT, in a manner comparable to the trilateration results, since all control points were treated as free and standard deviation was assigned to all of them (Table 2).

id	X	Y	Z	σX	σY	σZ	σXY	Rays
1	18.212	28.804	-26.363	0.0018	0.0006	0.0020	0.0019	5
2	22.531	36.492	-27.128	0.0010	0.0020	0.0018	0.0022	6
3	28.716	36.109	-27.483	0.0005	0.0019	0.0014	0.0020	15
4	32.265	34.202	-27.417	0.0011	0.0016	0.0015	0.0019	13
5	27.778	20.6	-26.333	0.0010	0.0013	0.0021	0.0016	3
6	25.164	14.483	-25.765	0.0008	0.0024	0.0028	0.0025	9
7	18.437	15.796	-25.543	0.0018	0.0021	0.0030	0.0027	7
8	29.837	24.854	-26.994	0.0006	0.0004	0.0010	0.0007	14
				0.0011	0.0015	0.0019	0.0019	

Table 2. Results from photogrammetry. All results in m.

After the adjustment, the overall image observation RMS is 0.8 pixels which corresponds to a sigma naught of 0.7 pixels, with a redundancy of ~410K and ~625K observations, for a total of ~72K of 3D tie points plus the 8 signalised points. The average intersecting angle is 26.1 degrees and 41.2 degrees for the 3D tie points and signalised points, respectively. The signalised point with maximum uncertainty is again #7 with σXY 0.03 m. The average σXY errors is 0.02 m and the average σZ error, is 0.02 m.

5. DISCUSSION AND CONCLUSIONS

The simulated trilateration scenario is rather favourable to actual practice, as it would need considerable resources and number of dives to be realised. In addition, it is unlikely that distances up to 20m can be measured at 27 m depth, and that they will be measured twice, not just because of currents or nitrogen narcosis, but because of limited visibility which renders communication among divers impossible. Also, the weighting scenario of 0.4% might also be questioned. On the other hand, the photogrammetry was performed with a mid-range DSLR camera with a low-cost lens, with photos taken in a single 20 min dive. The adjustment computation adopted a realistic standard deviation of 1 pixel in image measurements, and a simplified blunder removal approach, representing an actual and rather unfavourable scenario for network establishment and measuring for an underwater archaeological excavation.

The standard deviations of the calculated coordinates of the network points, after a minimally constrained network

adjustment with simulated trilateration and actual photogrammetry, show that photogrammetry may achieve three times better horizontal precision than trilateration, in this particular case.

In terms of vertical precision, it was proved that results of trilateration are worse than photogrammetry, a result which is expected both from photogrammetric and archaeological community. Photogrammetry is by definition a fully three-dimensional technique; oblique photos might be used to strengthen the camera network geometry and self-calibration. Despite the fact that, in this particular case, there were no oblique photos, photogrammetry outperformed trilateration. This is because the higher uncertainty in Z achieved through trilateration is caused by an almost 2D network with very small depth variation. This phenomenon is already known to practitioners and the main reason why dive computer depth readings are preferred to network height adjustment by the underwater archaeological community.

In short, photogrammetric network adjustment proved to be significantly better to trilateration adjustment, in this case. The ease and speed of data acquisition, along with the affordability and accessibility of underwater cameras, render photogrammetry a much better choice than trilateration for underwater network adjustment, in archaeological excavations. However, it should be pointed out that unmodelled systematic effects may still affect the accuracy of the photogrammetrically derived products. Such effects can be detected only by establishing a suitable control reference, which is, till now, a remaining challenge for underwater applications where high accuracy is needed. Future work must focus on determination of standard deviation of underwater tape measurements, and consecutive weighting scenarios, statistical testing of goodness of fit and outlier detection in both methods.

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