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Recovery and Readjustment of Historical Ocean Coast Control Stations in Oregon

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Abstract:* *In 1967, as directed by the Oregon Beach Bill, the Oregon State Highway Department undertook a survey of the entire coast to delineate the shore zone boundary. Survey control points and photo control were established for an aerial survey that same year, but only hardcopy records currently exist. If the 1967 survey coordinates can be accurately updated, the aerial imagery can be processed in modern photogrammetric software to produce digital elevation models of the entire coast, which will enable volumetric coastal change analysis over a period of half a century. The goal of this study was to develop and test a procedure for combining historic and current horizontal traverse survey data to update the 2D control survey coordinates (with the intent to add leveling data and extend the procedures to obtain 3D coordinates in a later study). First, a custom workflow and algorithms were developed to convert the historical

survey records to machine-readable format. GNSS data—both static post-processed and real-time network (RTN)—were then acquired for recoverable marks. An adjustment of the traverse data constrained to RTN coordinates was compared against an independent adjustment of the static GNSS data performed in the National Geodetic Survey OPUS-Projects software. The results show that the methods can produce updated horizontal coordinates for the 1967 survey accurate to within two centimeters.

Keywords: Oregon Beach Bill, survey control, OPUS-Projects, network adjustment

Introduction

Historical Background

In 1967, the 54th Oregon Legislative Assembly passed the Beach Bill (House Bill 1601), which was subsequently signed into law by Governor Thomas McCall. The Beach Bill codified the right of public access to Oregon beaches, with the aim of preserving "such public easements as a permanent part of Oregon's recreational resources" (Díaz Méndez 1999; Fifty-fourth Oregon Legislative 1967; Johnson and Schell 2013). With regard to the shore zone boundaries, the 1967 Beach Bill made reference to two elevation contours (relative to the National Geodetic Vertical Datum of 1929 (NGVD29), 1947 adjustment): the 4.9-m (16-ft) contour and the 1.7-m (5.7-ft) contour, from which the boundary was to be established 91 m (300 ft) inland in low-lying areas (streams, estuaries, rivers and creeks). Also included in the legislation was a directive to the State Highway Commission to "survey the land on the shore of the Pacific Ocean from the Columbia River on the north to the Oregon and California state line on the south for the purpose of locating the boundaries of the area zoned…" The State Highway Commission was further directed to complete the survey and present a report to the 54th Legislative Assembly Section the following year.

A shore control and pre-mark survey was performed by the Oregon State Highway Department, predecessor to the Oregon Department of Transportation (ODOT), followed closely by a photogrammetric survey. The control and pre-mark survey was conducted by a two-person reconnaissance crew, three-person traverse party, and two-person leveling party, all working long hours to meet the deadline of completing the survey within the 1967 field season (Jeter 1969). Pre-marks for the aerial photography (Fig. 1) established along the beach were generally logs or timber arranged in T- or cross-shaped patterns, while those on state highways were temporary striping material (Jeter 1969). Each end of each traverse line was tied to second-order, or better, horizontal control, which was either existing control (e.g., U.S. Coast and Geodetic Survey control stations) or established by the reconnaissance crew.

The equipment used by the traverse party included a 1 arcsecond theodolite, two targets, two subtense bars, various lengths of survey chains, and an Electrotape (Jeter 1969). Manufactured by Cubic Corporation, the Electrotape (Fig. 2) was an electronic distance measurement instrument (EDMI), which operated at microwave wavelengths, making it somewhat “all-weather.” It was reported to have a maximum range of up to 30 miles (50 km), with a reported accuracy of 1 cm ± 1 part in 300,000 (Breed and Hosmer 1966).

The aerial photography was acquired in sections, following closely behind the traverse and pre-mark field survey crews (Jeter 1969). Based on unpublished information available from the Oregon Parks and Recreation Department (OPRD), the aerial photography was captured using a Zeiss RMK A 15/23 camera with a Zeiss Pleogon A lens, with a nominal 6 inch (152 mm) focal length and 9×9 inch (230×230 mm) format, with the acquisition occurring between June and October, 1967. The nominal flight altitude was 3000 feet (900 m), yielding 1:6000 scale photography, and a standard endlap of 60% was used (OPRD, unpublished records).

The existing records maintained by OPRD include the original survey field notes and hardcopy printouts from a compass rule adjustment of the data, which was performed on IBM computers sometime around 1968 (Fig. 3, top and bottom, respectively). Prior to this study, these records had been scanned by OPRD, but were only available as image-format files, rather than machine-readable text. Challenges in conversion of the records to machine-readable text included the handwritten notes and comments, artifacts (e.g., smudges, extraneous marks), and general degradation of the paper records that had occurred since 1967-68.

Study Goals

In the half century since the Beach Bill was passed, significant sea cliff erosion, armoring, and other coastal change has occurred along the Oregon coast (Allan et al. 2013; Daniels et al. 1998; Ruggiero et al. 2013). If accurate coordinates, relative to current geodetic datums, can be obtained for the shore control stations established and/or used in the 1967 shore control survey, they can be used in photogrammetry software—either structure from motion/multiview stereo (SfM/MVS) or conventional softcopy photogrammetry—to create orthomosaics and digital elevation models (DEMs) for the entire coast. It will then be possible to difference the DEMs from current coastal lidar data to perform a detailed, quantitative volumetric change analysis for the Oregon coast over a period of half a century. Additionally, the updated coordinates are anticipated to be of use in other coastal surveying and mapping projects and to assist in establishing a bridge between historic and current coastal surveys.

In theory, it should be possible to obtain updated coordinates for the 1967 shore control through the following three-step process:

1. Apply the appropriate scale factor to convert from the original Local Datum Plane (LDP) coordinates (Armstrong 2017) to State Plane Coordinate System of 1927 (SPCS27) coordinates.
2. Use the National Geodetic Survey (NGS) Coordinate Conversion and Transformation Tool (NCAT) to perform a datum conversion from NAD 27 to NAD 83(2011).
3. Use NGS's Horizontal Time-Dependent Positioning (HTDP) utility to apply horizontal velocities to account for crustal motion between 1967 and the present.

However, when tested by OPRD, this three-step procedure was found to produce coordinates that differ from current Oregon Real-Time GNSS Network (ORGN) derived coordinates by up to 2 m (OPRD, personal correspondence). These differences are too large to be attributed to uncertainties introduced in the datum conversion and/or in the horizontal displacements applied in HTDP. Based on a review of the survey procedures and original records, it is likely that a major factor in the poor coordinate quality was error in some of the control to which the 1967 traverse lines were tied, while the traverse observations themselves (angle and distance measurements) appear to be generally free of blunders and large systematic errors, and, hence, accurate to within the limits of the 1960s-era surveying equipment used.

Based on these considerations, the primary goal of this study was to investigate the ability to use new GNSS observations and the historic traverse data in a combined network adjustment to obtain accurate, updated coordinates for the 1967 traverse stations. The methods were developed and tested using data from the Yaquina Bay (YB) traverse line in Newport, Oregon. A key consideration was the efficiency of the methods, based on the long-range goal of applying the methods to the 1967 traverse data for the entire Oregon coast. Additionally, the procedures developed and tested in this work were designed to be general enough to be applied by other researchers and practitioners interested in combining historic and current survey data. While the focus in this study was specifically on planimetric (2D) coordinates (in keeping with the 1967 procedures, in which the horizontal survey preceded the leveling), we discuss the extension to 3D using two methods: 1) incorporation of current lidar data on persistent features, and 2) a planned follow-on study, which will investigate a fully 3D adjustment, incorporating the leveling data.

Methods

Overview

The methods developed and tested in this study are depicted graphically in the workflow diagram in Fig. 4. First, an optical character recognition (OCR) approach was applied on the 368 hardcopy printout pages of the traverse adjustment from the 1967 survey to create digital survey data records. Next, recoverable stations from a traverse line were occupied with two different GNSS survey methods: Oregon Real-time GNSS Network (ORGN) 30-second to several minute occupations, and longer-occupation (> 2 hr) static observations for validation. Together with the 1967 survey observations, the station coordinates from the ORGN GNSS survey were input into MicroSurvey STAR*NET for performing a least-squares adjustment. The static occupations were post-processed using the NOAA National Geodetic Survey (NGS) Online Positioning User Service (OPUS) and adjusted using OPUS Projects. To assess the output of the approach, the coordinates adjusted in STAR*NET were compared with the coordinates processed in OPUS Projects. The individual steps in this workflow are described in the following sections.

The 1967 survey data includes a total of 28 traverse lines, with the number of survey stations varying from line to line. A total of 368 hardcopy survey sheets were scanned to create image-format files. Fig. 5. shows an example of the scanned YB traverse line survey sheets. The survey sheets consist of six measurement columns: Station, Distance, Deflection Angle, Bearing, Latitude, and Departure. The columns listed as Latitude and Departure are not the differences in northing and easting between the successive stations comprising each course, but, rather, the Local Datum Plane (LDP) northings and eastings of the stations. The LDP coordinates are obtained by scaling SPCS27 coordinates to a plane representing the mean ground elevation, a procedure that was commonly used by ODOT and its predecessor, the State Highway Department, prior to the establishment of Low Distortion Projections (Armstrong 2017). The units of Distance, Latitude, and Departure are U.S. Survey Feet (sft) and were converted to meters in SPCS 83 for adjustment in this study. The deflection angles and bearings are in degrees-minutes-second (D-M-S) format. Following convention, the deflection angles are measured from the prolongation of the preceding course to the next course, either counterclockwise (left, denoted by "L" or "-") or clockwise (right, denoted by "R" or "+"). The bearings are given in the specified quadrant, recorded as "NE", "NW", "SE", and "SW" in the measurement column. In the 1967 survey sheets, once the initial bearing was observed, the remaining bearings were calculated sequentially by adding the deflection angle (accounting for the algebraic sign associated with "L" and "R" angles) to the previous bearing measurement without additional observations.

Historic Survey Data Recognition and Preprocessing

The next step was to apply an optical character recognition (OCR) technique to convert the scanned survey sheets into a digital (machine readable) form suitable for ingestion into any number of survey adjustment software packages. While the OCR portion of this study was initially envisioned to be a minor aspect of the project, it proved to be highly challenging, and hence, is discussed in some detail here. Initially, OPRD attempted to run commercial-off-the-shelf (COTS) OCR software on the scanned records, but with limited success. The fact that the scans consisted of surveying records resulted in both complications and benefits, as compared with more typical OCR tasks. A major complication is that a single misrecognized character can result in a blunder that, if undetected, will propagate to large errors in computed coordinates. (This is in contrast to, say, scans of historical texts, in which a misrecognized character can be corrected later when considered in context of the word, sentence, or paragraph in which it appears, and, even if undetected, might not inhibit the interpretation of the text.) However, on the positive side, the fact that the records could be checked using equations of plane surveying proved to be a major advantage for detecting such errors. Based on these considerations, the custom workflow described below, was tested and implemented.

A Tesseract OCR program (Smith 2007) implemented in MATLAB was first used to recognize the survey records from the scanned sheets. The output files were in comma-separated values (CSV) format, storing the six types of survey measurements in separate columns. However, as shown in Fig. 5, the scanned survey sheets included significant noise and artifacts, such as handwritten letters, horizontal line separators, printing or binding holes, and smudges, rendering it difficult to recognize. Therefore, a set of preprocessing steps, including image cropping, median filtering (using a 5×5 filter window, a size determined empirically), binarization using Otsu's method (Otsu 1979), morphological filtering, and column separation, was applied to reduce the noise and improve the recognition rates of the OCR process. The outputs of each of these preprocessing steps are shown in Fig. 6, and the column separation, which enabled reduction of noise and artifacts, is illustrated in Fig. 7.

The recognized and digitized survey measurements were found to still contain errors, due to residual noise in the image. Fig. 8(a), case 2 (delineated by a dashed box), shows an example of the over-detection due to handwriting between the survey measurement rows, while Fig. 8(a) cases 1 and 3 show examples of the incorrect detection, due to inadequate noise reduction. These errors could potentially be reduced further for a single image by carefully adjusting the preprocessing parameters. However, this parameter tuning is a challenging, labor-intensive process, due to needing to account for scan-to-scan variations. Therefore, semi-automated refinement was performed, leveraging the confidence values (on a unitless scale of 0-1) output by the OCR program. Any words containing a character with a confidence value < 0.7 were highlighted [Fig. 8(a)] to help the user identify the location of over- or incorrect detections in CSV files.

Even after the refinement, errors were found to still exist. However, an important constraint is provided by the fact that these are not arbitrary data records, but, rather, are plane surveying records. Hence, we can use plane surveying formulas as a check on the digitized records and as a means of catching remaining OCR errors. Specifically, the following mathematical conditions are assumed to hold, if the digitized records are recognized correctly:

$$\sqrt{\left(\Delta N_{ij}\right)^2 + \left(\Delta E_{ij}\right)^2} = d_{ij} \tag{1}$$

$$\tan^{-1}\left(\frac{\Delta E_{ij}}{\Delta N_{ij}}\right) = \beta_{ij} \tag{2}$$

$$\beta_{ij} \pm \delta_{ij} = \beta_{jk} \tag{3}$$

where ΔN = latitude (difference between consecutive northings), ΔE = departure (difference between consecutive eastings), *d* = distance (or, length of line), β = bearing, δ = deflection angle, and indices i, j and k denote consecutive records (i.e., $j = i + 1, k = j + 1$). Eq. 1 expresses the condition that the latitude and departure of the line, summed in quadrature, should give the length of the line. Eq. 2 expresses the condition that the inverse tangent of the ratio of the departure to the latitude of the line should give its bearing (accounting for algebraic signs and quadrant). And, Eq. 3 expresses the condition that the bearing of the current line added to the defection angle (accounting for quadrant and algebraic sign) should equal the bearing of the next line.

Table 1 and 2 show an example of the evaluation with two stations (YB3 and YB4) and their measurements. The stations that do not meet the above conditions were highlighted in green in the CSV files to help detect and correct the remaining errors. In this research, a total of 28 traverse lines were digitized from the 368 scanned survey sheets and processed to generate CSV files, a process which took approximately one month to complete. An example of the output is shown in Fig. 8[b].

Experiment

Among the 28 traverse lines, the YB (Yaquina Bay) transverse line (Fig. 9) was selected for testing the combined adjustment of historic and current survey data. The YB transverse line includes 64 distances and 63 deflection angles recognized from the digitized survey sheets. From the 67 stations of the YB traverse line, four stations, YB77, YB19, YB32, and YB1000, were selected as suitable for recovery and occupied with GNSS. For each station, three static GNSS occupations of > 2 hours were performed. RINEX observation files were then uploaded to NGS OPUS and then OPUS Projects for post-processing. Additionally, receivers were configured to receive Oregon Real-time GNSS Network (ORGN) Master-Auxiliary Concept (MAC) corrections (ODOT 2020; Weaver et al. 2018), with occupation times ranging from 0.5 to 5 minutes. Each station was occupied three times on separate days with time spacings between occupations ranging from weeks to months, based on field and equipment schedules. The average values of the ORGN-derived coordinates were used later as fixed stations for adjustment in STAR*NET. Table 3 lists the four stations' SPCS83 Oregon North (NAD 83 (2011)) average coordinates in units of meters and their standard deviations. Fig. 9 shows the distributions of the four recovered stations overlaid on Esri World Imagery in ArcGIS, while Fig. 10. shows the monumented control points of the four stations.

An interesting example of the types of questions that arise when working with historical survey data is provided by the photo of station YB1000 (also known as SBP 10) in Fig. 10. The year stamped on the mark is 1968, while the control survey was completed in 1967. All available datasheets for this mark were reviewed in an attempt to resolve this discrepancy, but no relevant information was found. While it is possible that the stamping was added at a later date, we believe it is more likely that the initial mark was less permanent (e.g., a PK nail or similar), and the mark was reset the following year using a bronze survey disk. It is reasonable to assume that appropriate care would have been taken in resetting the mark, but this does introduce the potential for additional uncertainty in the mark's coordinates.

OPUS Projects

OPUS Projects is a web-based tool for visualization, management, processing, and sharing for geodetic network solutions by baseline processing of multiple GNSS occupations (Armstrong et al. 2015; Gillins and Eddy 2017). It consists of three steps: (1) OPUS upload; (2) session processing; and (3) network adjustment. First, the user needs to create a project through OPUS Projects (NGS 2020a). Uploading data to the project is achieved through OPUS (NGS 2020b), which enables the user to tie local surveys to the National Spatial Reference System (NSRS). The user uploads raw GNSS data, which is processed by NGS software using the NOAA Continuously Operating Reference Station (CORS) network to obtain NSRS coordinates, typically at an accuracy of a few centimeters. The coordinates of the OPUS solutions are then linked to the project created by the user and used as the a priori input for session processing in OPUS Projects.

The session processing forms baselines between the simultaneously-observed project marks and CORSs in proximity, which are processed together to increase consistency between them. The CORS Network Design strategy automatically selects all CORSs included in the OPUS submissions, among which one CORS (p367) is chosen as a “hub” that is selected as the common station for all baselines. Fig. 11 shows the baseline network formed with the static observations for YB77, YB19, YB32, YB1000 (depicted as circles), and the CORSs (circle containing a triangle) in the OPUS Projects tool. It is recommended to add a distant CORS between 350 to 800 km from the hub for tropospheric correction (Armstrong et al. 2015; Gillins and Eddy 2017). In this project, a distant CORS, P394, that is nearly 500 km away from the hub was added to the network, as shown in Fig. 11. In the preferences tab, the user can set the “Data & Solution Quality Thresholds” to identify the project mark processing results that do not meet the conditions. The default preference setting provided by the OPUS Projects was adopted in this research. Once the session process is finished, data files whose processing results exceed the thresholds can be visually identified by their icon style and color changes, as shown in Fig. 12.

Based on the session processing results, the user can select the sessions that meet the predefined preferences for input into the next adjustment step, in which interlinking sessions are adjusted to increase accuracy using a least-squares network adjustment. In this research, six session results (2018-095A, 2019-274A, 2019-274B, 2019-345A, 2020-003A, and 2020-009A) were selected for the inputs for the adjustment according to the session results in Fig. 12. NGS CORS p367 was constrained in the final network adjustment. Table 3 lists the coordinates for YB77, YB19, YB32, and YB1000 marks adjusted by OPUS Projects, along with their differences with respect to ORGN-derived coordinates. Uncertainties are not provided for OPUS Projects coordinates because it is known they are optimistic by nearly an order of magnitude (Schenewerk 2020). (This known issue had not yet been addressed in the production version of OPUS projects at the time of this study.) Based on this known bias, the estimated uncertainty of the OPUS Projects adjusted coordinates is ~1 cm, which is consistent with the RMS of 1 cm for the OPUS network adjustment.

Because the ORGN is aligned with NGS CORSs (ODOT 2020), OPUS-adjusted marks serve as independent checks for validation of the STAR*NET results, which are consistent within ~2 cm. The reason for the systematic mean difference of 2 cm in northing and 0.6 cm in easting is not known, although it may be due at least in part to how the ORGN is aligned with NGS CORSs. Not all base stations in the ORGN are NGS CORSs, and only CORSs with velocities rigorously computed by NGS are constrained for determining ORGN base coordinates (ODOT 2020). In addition, the coastal region of Oregon is an area of known crustal deformation adjacent to a subducting tectonic plate boundary. Nonetheless, the 2-cm horizontal agreement between the ORGN and OPUS Projects is quite good, and is comparable with the estimated accuracy of both the ORGN and OPUS Projects coordinates.

Least Squares Adjustments

Adjustments were performed in the commercial software package, MicroSurvey STAR*NET. The first step was to establish the stochastic model, which enables appropriate weighting of the observations in a least squares adjustment. The standard errors of distance observations made with the Electrotape were taken to be 0.01 m ± 3 ppm, based on the published accuracy of the Electrotape (Breed and Hosmer, 1966). Centering error of ±2 mm was assumed for distance observations made with the Electrotape. Unfortunately, the 1967 survey notes did not indicate which distance measurements were taped and which were made with the Electrotape. However, a review of the available records indicated that the Electrotape was generally not used within the traverses; rather, it appears to have been used primarily for tying into second order (or better) control at either end of a line, and for longer distance measurements or those that could not be taped (e.g., due to being over water). Meanwhile, according to Jeter (1969), the subtense bars were used only occasionally. Accordingly, for the YB traverse, all distances were assumed to be taped, except the last course, YB32 - YB1000, which had an observed distance of 846.216 m and crossed the mouth of Yaquina Bay, and, hence, would have been a good candidate for use of the Electrotape.

The accuracy of taped distances (with appropriate corrections applied), was taken to be 1:10,000 (Afeni, 2011), and, hence, the standard errors of the taped distances were set to ±100 ppm in STAR*NET. With the assumption that error in plumbing the tape is accounted for in the ±100 ppm, the centering error was set to zero for taped distances. While the theodolite used in the survey was listed as a 1" instrument (Jeter, 1969), this was taken to be the least count of the horizontal circle, rather than the "DIN accuracy" (Professional Surveyor, 2002), and ±8" was used as the standard error of the deflection angle measurements. This value was initially estimated as ±10", but that was found to be too pessimistic, based on preliminary least squares adjustments of the traverse. Holding all other a priori standard error estimates constant, a deflection angle standard error of ±8" was the integer value that yielded an adjustment reference standard deviation closest to 1. Table 4 lists the a priori error estimates used for the adjustments.

The functional model (Mikhail, 1976) implemented in the adjustment was based on the plane geometry model of a link traverse with horizontal angle and distance observations. The YB traverse line included a total of 67 stations. Among these, two stations (YB88 and another, unnumbered station) were excluded from the adjustment due to the lack of distance measurements. The adjustment thus included 64 distances and 63 deflection angle measurements, and the average values of three ORGN observations for YB77, YB19, YB32, and YB1000 stations listed in Table 3 were used as constraints. In the fully-constrained adjustments, the coordinates of the stations with the independent observations were set to 'FIXED' in STAR*NET, so that they received no corrections (Starplus, 2004).

Results

The effects of the number and distribution of the fixed stations on the adjustment results were investigated. Table 5 contains the differences between the coordinates adjusted from STAR*NET and OPUS Projects. Each row indicates the different combinations of the fixed stations (highlighted in bold) and the remaining stations that were used for validation to evaluate the adjusted coordinates. Note that when adjusting the network with one fixed station, it is necessary to have at least one bearing connected to the fixed station, which can be estimated by the Condition (2) in Table 2.

The differences in easting (ΔE) were generally larger than those in northing (ΔN), because of the long stretch of the YB traverse line in the north-south direction. As anticipated, increasing the number of fixed stations tends to decrease the differences between the STAR*NET and OPUS Projects results. However, regardless of the number of fixed stations, the differences of YB77 station are still large (> 2 m in easting), because it is relatively distant from the other three fixed stations [Fig. (9)]. This indicates that it is necessary to include YB77 as the fixed station to ensure a reliable adjustment. The smallest difference (< 0.02 m in easting and northing) was achieved on YB32 when using YB77, YB19, and YB1000 as fixed stations because this combination enables the most evenly-distributed fixed stations across the YB traverse line as shown in Fig. 13. With YB77, YB19, and YB1000 as fixed stations, all the remaining stations in the YB traverse line were readjusted in the State Plane Coordinate System of 1983 (SPCS83), zone code 3601: Oregon North, NAD 83(2011), with units of meters.

Table 6 shows the statistical summary from analyzing the results of the adjustments in STAR*NET. In the final, fully-constrained adjustment, the Chi-square test was passed at the 0.05 significance level. The “error factor” (reference standard deviation) was 1.084. The analysis of the adjustment results as well as the statistics summarized in Table 6 provided indication that the functional and stochastic models were set appropriately.

Discussion

Based on the results of this study, including both the quantitative results presented above and the experiences of the study team in performing the data analysis and processing, several recommendations can be made. The use of a real-time GNSS network (the ORGN) was found to be highly beneficial in reducing the data acquisition time. The results of the different adjustments demonstrate the importance of obtaining new GNSS data for a minimum of three stations per traverse, distributed as evenly as possible over the spatial extent of the traverse, including at either end of the line. Based on related research conducted concurrently by the OSU geomatics group, it is recommended that the repeat occupations be separated in time by at least one hour (Simpson et al. 2020) and that two separate ORGN occupations of three minutes (Allahyari et al. 2018) be performed. If this is not logistically feasible (e.g., due to requiring extended traffic control), the procedure described by (Gillins et al. 2019) can be used to obtain independent, back-to-back measurements. Specifically, the GNSS antenna should be removed and inverted, such that it loses initialization and must reinitialize. Ideally, the centering over the occupied point should also be repeated.

Another consideration in selecting marks for GNSS occupation is that any crustal motion having occurred between historic survey and the present must have resulted in approximately equal horizontal displacements to each of the traverse line stations occupied with GNSS and their neighboring stations. This condition would likely be violated if, for example, one of the stations was in an area impacted by a landslide between 1967 and the present. In this case, it would not be possible to use the new GNSS survey data and 1967 traverse observations together in an adjustment and expect to obtain good results, due to the change in the traverse geometry.

In determining which stations still exist and are suitable for GNSS occupation, Google Street View was found to be a useful office tool, prior to searching for the stations in the field. An iterative process can be used in locating additional marks: once the first adjustment is completed, the improved coordinates can provide better starting locations for locating additional stations. If MicroSurvey STAR*NET is used to perform the least squares adjustment of the 1967 traverse data and new GNSS data, it is recommended that the parameter values listed in Table 4 be used as initial values and refined, as needed, to obtain a reference factor close to 1.0 with a passing result for the Chi-square test.

Conclusions

The results of this study demonstrate the feasibility of using historical survey observations with new GNSS observations in a combined network adjustment to obtain accurate coordinates relative to modern geodetic datums. Additional contributions of this research include the development and testing of procedures for: 1) converting historical, hardcopy survey records to machine-readable format for use in modern surveying software, including error checking leveraging plane surveying formulas; and 2) collecting GNSS data on recoverable stations to use in the combined adjustments. While these are significant first steps towards our long-range goal of obtaining updated coordinates for the entire 1967 survey of the Oregon coast and using the updated coordinates in volumetric change analysis, this study also served to identify important topics to investigate in follow-on research.

One recommendation for continuing research is to investigate possible methods of increasing the number of 1967 control stations that can be used in the adjustment. The process of iterative adjustment and mark recovery is likely to be useful for many of the traverse lines, as the improved coordinates from the first round(s) of adjustment may enable additional stations to be located. Stations that cannot be directly occupied with GNSS may still be able to be used, if two temporary control stations can be established nearby and coordinates of the 1967 control station obtained via total station measurements from the temporary control stations.

It is also important to note that this project focused strictly on horizontal coordinates of the shore control stations, in keeping with the 1967 survey, in which the horizontal survey was performed and adjusted (via a Compass Rule adjustment) separately from the leveling survey. The updated 2D coordinates from this study are anticipated to be of great value in generating accurate orthomosaics, and we also intend to investigate the use of current lidar-based elevations of persistent features (e.g., concrete surfaces that have remained in place and unchanged since the late 1960s) as vertical-only control points.

However, it is also important to note that the 1967 survey also included third order leveling run along the traverse and to the pre-parks on the beach to obtain elevations on all stations (Jeter, 1969). Therefore, an important, planned extension of this work is to perform a fully 3D combined adjustment of the 1967 traverse observations, leveling data, and recent GNSS observations.

This research, enhanced through the recommended follow-on studies, is expected to enable use of the updated coordinates on the aerial photography pre-marks to generate accurate orthomosaics and DEMs from the 1967 imagery using modern photogrammetry software—either structure from motion (SfM)/multi-view stereo (MVS) software, or conventional softcopy photogrammetry software. From these DEMs and modern lidar data, such as the Joint Airborne Lidar Bathymetry Technical Center of Expertise (JALBTCX) coastal lidar data collection completed in summer 2020, it should be possible to perform rigorous coastal change analysis covering a period of over half a century. Furthermore, it is not necessary to limit the change analysis to simply "end-point rates"; rather, change can be more rigorously evaluated over time using additional aerial photography collected between 1967 and the present. Based on OPRD records, aerial photography was acquired for portions of the Oregon coast in 1968, 1969, 1972, and at various times in the 1980s. To facilitate use of the procedures developed in this research by others interested in combining historic and current survey data in a new adjustment, the source code and detailed procedures will be made publicly available.

Data Availability Statement

The measurements and adjusted coordinates of the YB traverse line are available at https://files.prd.state.or.us/s/fi3xLzmGzCpCpm8. Additional data from this project will be made publicly available by OPRD at the completion of the research project.

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Figure Caption List

Fig. 1. Pre-marks for aerial photography on (a) state highway and (b) on beach. Source imagery is the property of the OPRD and available publicly at: https://arcg.is/qzWHi0.

Fig. 2. Cubic Corporation Electrotape microwave EDMI (Copyright Cubic Corporation, used with permission 2021).

Fig. 3. (a) Original field survey notes and (b) hardcopy printouts from a compass rule adjustment of the data performed around 1968 on digital computers. Both exhibit the poor quality typical of these hardcopy records. (Images reproduced from OPRD unpublished records with permission from the OPRD.)

Fig. 4. Workflow diagram illustrating the key steps in the procedure developed and tested in this study.

Fig. 5. Example of a scanned survey sheet from the Yaquina Bay (YB) traverse line. The presence of handwritten notes, smudges, printer tractor-feed holes, and other artifacts is typical of these scanned sheets (Images reproduced from OPRD unpublished records with permission from the OPRD.)

Fig. 6. Preprocessing steps: a) image cropping; b) median filtering; c) Otsu binarization followed by a morphological opening process; and d) column separation method (Images reproduced from OPRD unpublished records with permission from the OPRD.)

Fig. 7. Example of column separation: (a) graph of the number of occupied pixels in columns of the binarized image and (b) normalized and thresholded graph.

Fig. 8. Example of (a) recognized and (b) refined characters in CSV format. In (a), the highlighted text has been automatically flagged by the software as requiring further evaluation. The numbered boxes in (a) are the cases discussed in the text.

Fig. 9. Yaquina Bay (YB) line traverse stations (green triangles) and those recovered and occupied with GNSS in this project (red triangles, labeled). (Map source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and GIS User Community.)

Fig. 10. Monumented control points: (a) YB77 (aka, LACK); (b) YB19; (c) YB32; and (d) YB1000 (aka, SBP 10).

Fig 11. Baseline network in OPUS Projects.

Fig. 12. Session process results. The marks that do not meet the specified data and solution quality thresholds (set to the default values in this study) are marked with a diagonal line.

Fig. 13. YB traverse line adjusted in STAR*NET using YB77, YB19, YB1000 as fixed stations.

Table 1. Example of measurement information

Station	Distance (m)	Deflection Angle (dms)	Bearing (dms)	Latitude (m)	Departure (m)
YB3	254.26 (834.19 ft)	005 05 50 L	S 08 19 25 W	120104.18 (394042.57 ft)	327352.69 (1073991.77 ft)
YB4	166.68 (546.86 ft)	000 00 00 L	S 03 13 35 W	119852.59 (393217.17 ft)	327315.88 (1073871.01 ft)

Table 2. Example of measurement evaluation

Condition	ΔN (m)	ΔE (m)	Condition (1) (m)	Condition (2) (dms)	Condition (3) (dms)
Calculation	394042.57 − 393217.17 = 251.58 (825.40 ft)	1073991.77 − 1073871.01 = 36.81 (120.76 ft)	$\sqrt{(\Delta N)^2 + (\Delta E)^2}$ = 254.26 (834.19 ft)	$\tan^{-1}\left(\frac{\Delta E}{\Delta N}\right)$ = S 08 19 25 W	$\beta - \delta =$ S 08 19 25 W − 005 05 50 L = S 03 13 35 W

Table 3. Averaged coordinates of ORGN field observations and adjusted coordinates by OPUS project (unit: meters).

Stations	Averaged ORGN observations		OPUS-adjusted coordinates		*OPUS minus ORGN*	
	Northing	Easting	Northing	Easting	*dNorth*	*dEast*
YB77	121698.938 ± 0.015	2217919.595 ± 0.003	121698.918	2217919.580	*-0.020*	*-0.015*
YB19	114988.526 ± 0.012	2217969.863 ± 0.005	114988.508	2217969.855	*-0.018*	*-0.008*
YB32	112685.409 ± 0.010	2217627.938 ± 0.003	112685.389	2217627.934	*-0.020*	*-0.004*
YB1000	111998.388 ± 0.009	2218122.041 ± 0.002	111998.364	2218122.044	*-0.024*	*0.003*
				Mean differences	***-0.020***	***-0.006***

Table 4. Project a priori error estimates for the adjustment in STAR*NET.

Equipment	Standard errors			Centering errors	
	Distance constant (meters)	Distance PPM	Angle (seconds)	Instrument (meters)	Target (meters)
Electrotape	0.010	3	-	0.002	0.002
Steel tape	0.000	100	-	-	-
Theodolite	-	-	8.000	0.002	0.002

Table 5. Validation of the coordinates adjusted in STAR*NET using ORGN observations (units: meters).

# of fixed stations	YB77		YB19		YB32		YB1000	
	ΔN	ΔE	ΔN	ΔE	ΔN	ΔE	ΔN	ΔE
1	***0.020***	***0.015***	0.193	-1.138	0.340	-2.318	0.132	-2.679
	-0.154	1.309	***0.018***	***0.008***	0.173	-1.222	-0.046	-1.599
	-0.315	2.794	-0.144	1.300	***0.020***	***0.004***	-0.213	-0.392
	-0.315	2.794	0.092	1.691	0.257	0.394	***0.024***	***-0.003***
2	***0.020***	***0.015***	***0.018***	***0.008***	0.090	-0.669	-0.010	-0.881
	0.020	***0.015***	0.029	0.371	***0.020***	***0.004***	-0.021	-0.125
	0.020	***0.015***	0.132	0.943	0.158	0.254	***0.024***	***-0.003***
	-0.178	-1.909	***0.018***	***0.008***	***0.020***	***0.004***	0.075	0.008
	-0.182	-2.375	***0.018***	***0.008***	-0.018	0.012	***0.024***	***-0.003***
	-0.158	-2.043	0.039	0.064	***0.020***	***0.004***	***0.024***	***-0.003***
3	***0.020***	***0.015***	***0.018***	***0.008***	***0.020***	***0.004***	0.083	0.019
	0.020	***0.015***	***0.018***	***0.008***	-0.015	0.013	***0.024***	***-0.003***
	0.020	***0.015***	0.033	0.145	***0.020***	***0.004***	***0.024***	***-0.003***
	-0.179	-2.005	***0.018***	***0.008***	***0.020***	***0.004***	***0.024***	***-0.003***

Note: Fixed stations are italicized and highlighted in bold

Table 6. Adjustment statistical summary in STAR*NET using YB77, YB19, YB1000 as fixed stations.

Fixed stations	Iteration	# of adjusted stations	# of unknowns	# of redundant observations	# of Observations			Sum squares of standardized residual			reference factor			Chi-Square test at 0.05 level
					Angles	Distances	Total	Angles	Distances	Total	Angles	Distances	Total	
YB77 YB19 YB1000	2	62	124	3	63	64	127	2.323	1.203	3.526	1.249	0.892	1.084	passed