الهيئة العامة للطرق
Roads General Authority

كـود الطرق السعـودي
SAUDI HIGHWAY CODE

SAUDI HIGHWAY CODE (SHC)

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Planning and Preliminary Studies
SHC 202 – Surveying and Mapping

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المملكة العربية السعودية
KINGDOM OF SAUDI ARABIA

كود الطرق السعودي

SAUDI HIGHWAY CODE

Key list of the Saudi Highway Codes

No.	Thematic Category	Code	Title
1	Introduction	SHC 101	General
2	Planning and Preliminary Studies	SHC 201	Planning Process
3		SHC 202	Surveying and Mapping
4		SHC 203	Preliminary Studies
5	Design of Highways, Bridges and Tunnels	SHC 301	Highway Geometric Design
6		SHC 302	Highway Facilities and Utilities Design - Hydrology and Hydraulic Design
7		SHC 303	Highway Facilities and Utilities Design Rest Areas, Truck Inspection Stations, Parking & Garage Facilities
8		SHC 304	Highway Facilities and Utilities Design - Passive Safety Systems Design
9		SHC 305	Highway Facilities and Utilities Design - Work Zone Design
10		SHC 306	Highway Facilities and Utilities Design - Public Utilities, Highway and Street Lighting, Control and Monitoring Devices
11		SHC 307	Highway Facilities and Utilities Design - Landscape Planting, Outdoor Advertising
12		SHC 308	Pavement Design
13		SHC 309	Material Specifications and Standardized Testing
14		SHC 310	Bridges and Tunnels Design
15	Construction of Highways, Bridges and Tunnels	SHC 401	Construction of Highways
16		SHC 402	Construction of Bridges and Tunnels
17		SHC 403	Construction of Highway Facilities
18	Highways, Bridges and Tunnels Maintenance and Management Systems	SHC 501	Pavement Maintenance Management Systems
19		SHC 502	Bridges and Tunnels Maintenance and Management Systems
20		SHC 503	Highway Facilities Maintenance and Management Systems
21	Traffic Engineering and Road Safety	SHC 601	Traffic Engineering
22		SHC 602	Manual on Uniform Traffic Control Devices
23		SHC 603	Road Safety
24	Environmental Aspects of Highways	SHC 701	Environmental Aspects of Highways
25	Autonomous Vehicles Requirements	SHC 801	Autonomous Vehicles Requirements

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1. Introduction

1.1. Summary of Chapters

Volume SHC 202 - Surveying and Mapping is divided into 6 Parts containing 21 chapters. Part I – Surveying, Mapping and Road Projects (Chapter 1 through 6) covers general requirements for surveying and how it is applied to Road Projects. Part II – TPS, Surveying and Leveling (Chapter 7 through 9) covers the specifics when using Total Point Stations (TPS) and Level instruments for surveying. Part III – GNSS Surveying (Chapter 10 through 12) provides the specifics when using Global Navigation Satellite System (GNSS) instruments for surveying. Part IV – Other Surveying Methodologies (Chapter 13 through 16) covers the requirements when using Aerial Surveying, Laser Scanning, Mobile Mapping, and Bathymetric surveying instruments and methodologies for road projects. Part V – Surveying Procedures for Road Projects (Chapter 17 through 20) provides the specifics for special surveying procedures employed in road projects. Part VI – Exception Procedure (Chapter 21) closes the code by providing the exception procedure.

A brief outline of these parts and chapters is given below:

Chapter 1. Introduction - This chapter provides an overview of the chapters, the scope of the volume, and a full list of the standards referenced within the volume.

Chapter 2. Basic Survey Concepts - This chapter introduces surveying and its basic methods, concepts, and principles as well as the definition of survey datum.

Chapter 3. Survey Requirements for Highway and Road Projects - This chapter describes the surveying procedures involved in road design phases, construction, and maintenance, as well as other basic concepts regarding the personnel involved in surveying.

Chapter 4. General Guidelines for Field Survey Work - This chapter describes planning and preparation procedures related to survey field work, as well as general directions for fieldwork applied in all surveying methodologies.

Chapter 5. Preparing Maps - This chapter describes the form of the final maps to be produced by surveying works.

Chapter 6. Data Delivery and Formats - This chapter provides information on the available data formats used when submitting final survey data.

Chapter 7. Horizontal Control (TPS) - This chapter describes the procedures required for establishing horizontal control in road projects with the help of Total Point Stations (TPS).

Chapter 8. Vertical Control (Leveling) - This chapter describes the procedures required for establishing vertical control in road projects with the employment of levels.

Chapter 9. Topographic Survey (TPS) - This chapter describes the procedures involved in ground surveying with Total Point Stations.

Chapter 10. GNSS Surveying - This chapter provides general information about GNSS surveys and techniques.

Chapter 11. GNSS Data Collection - This chapter describes the processes required for GNSS surveys in the field.

Chapter 12. GNSS Data Processing - This chapter describes the procedures involved in GNSS data processing and error adjustment.

Chapter 13. Aerial Surveying - This chapter describes the procedures of aerial surveys, introducing the concept of photogrammetry and LiDAR aerial surveys with manned and unmanned aerial vehicles.

Chapter 14. Laser Scanning - This chapter describes the procedures required for using Terrestrial Laser Scanning (TLS) in road projects.

Chapter 15. Mobile Mapping - This chapter describes mobile mapping surveys along with the introduction to the procedures, data processing, and equipment needed.

Chapter 16. Bathymetric Surveying - This chapter describes the procedures employed for using bathymetric, echo-sounding surveys in road projects.

Chapter 17. Digital Terrain Modelling -This chapter provides guidelines for Digital Terrain Model (DTM) data collection, creation, and quality control.

Chapter 18. Cadastral for Road Projects - This chapter covers the tasks involved when creating cadastral maps for road projects.

Chapter 19. Construction Surveys and AMG - This chapter sets the necessities for construction surveys and the employment of Automatic Machine Guidance (AMG) in the construction of road projects.

Chapter 20. Road Maintenance and BIM - This chapter covers the notion of Building Information Model (BIM) when employed in road projects.

Chapter 21. Exception Procedure - This chapter provides the Design Exception Procedure required to override the content of this code.

1.2. Scope

All survey work or survey-related activity performed by all relevant entities is subject to the policies and procedures of this Volume.

This section is designated as a User Manual. It defines currently accepted and generally established rules, appropriate surveying methods, and equipment for accurately conducting mapping activities for highway projects. The content of this section is binding, but exceptions in specific cases are allowed if they are properly supported and approved following the Design Exception Procedure provided in the corresponding Chapter 21. The content of this section forms the basis of contracts for the competent road authorities.

1.3. Reference Standards and Codes

Standards for all geodetic instrument calibration and procedures shall be as specified in their specifications, in the Contract documents, if any, and the following, in their latest edition:

- ISO Standards.
- IEC standards.

Table 1-1 presents the International Standards Organization (ISO) and International Electrotechnical Commission (IEC) standards related to surveying and mapping, including designations and titles.

Table 1-1 Designations and Titles for ISO and IEC Standards Applying to Volume SHC 202

ISO	IEC	Title
12858-1		Optics and Optical Instruments – Ancillary Devices for Geodetic Instruments - Part 1: Invar Levelling Staffs
12858-2		Optics and Optical Instruments – Ancillary Devices for Geodetic Instruments - Part 2: Tripods
12858-3		Optics and Optical Instruments – Ancillary Devices for Geodetic Instruments - Part 3: Tribrachs
16331-1		Optics and Optical Instruments – Laboratory Procedures for Testing Surveying and Construction Instruments - Part 1: Performance of Handheld Laser Distance Meters
17123-1		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 1: Theory
17123-2		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 2: Levels
17123-3		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 3: Theodolites
17123-4		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 4: Electro-Optical Distance Meters (EDM Measurements to Reflectors)
17123-5		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 5: Total Stations
17123-6		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 6: Rotating Lasers
17123-7		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 7: Optical Plumbing Instruments
17123-8		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 8: GNSS Field Measurement Systems in Real-Time Kinematic (RTK)
17123-9		Optics and Optical Instruments – Field Procedures for Testing Geodetic and Surveying Instruments - Part 9: Terrestrial Laser Scanners

ISO	IEC	Title
	60825-1	Safety of Laser Products - Part 1: Equipment Classification and Requirements

2. Basic Surveying Concepts

2.1. Introduction to Surveying

Mapping and Surveying also referred to by the term Geomatics, is the discipline of gathering, storing, processing, and delivering geographic or spatially referenced information; and is principally concerned with understanding the shape and properties of the physical world. Due to its fundamental relationship with natural and man-made features of the environment, surveying works play a very important role in every construction project, human security, and welfare. Surveying serves as the basis of the majority of all construction studies that we see today, especially as regards the organization of cities, the establishment of public utilities, and urban and rural planning. Therefore, surveying and mapping play an important role in every aspect of road-related activities such as road design, construction, and maintenance.

2.2. Definition of Surveying

Surveying is the science and art that aims at representing the Earth's surface, including its horizontal and vertical features, on maps with appropriate drawing scales; surveying indicates all existing contents on this surface, whether they are natural, such as mountains, valleys, rivers, seas, or man-made, such as buildings, roads, and the boundaries of private and public property.

Surveying works are based on measuring distances, angles, directions, and heights using optical, electronic, and digital monitoring devices of multiple accuracy and purpose. It is also based on office work in which various types of calculations, adjustments, and drawings are required through computers and interactive workstations.

For the purpose of this code all "surveying" mentioned hereafter shall be referred to as "Road Surveying".

2.3. Terrestrial Surveying

Terrestrial surveying determines the horizontal and vertical locations of points and features on the surface of the earth according to a given reference system; it also sets directions and measures lengths, defines land boundaries, and finally produces detailed maps of various scales describing land features details. Terrestrial surveying can be divided into two parts:

- Planimetric Surveying: It is the area that aims to represent the surface of the earth as a flat limited surface regardless of its spherical shape and its use is limited in the surveying and staking of small areas. It is used in the study and implementation of engineering projects of limited size, such as buildings, bridges, tunnels, and mines.
- Geodetic Surveying: It is the area that examines the representation of the earth's surface as an irregular, near elliptic, surface. It is used in large areas such as countries and continents, and sizeable projects, such as road projects. It aims to determine the shape, size, and gravity of the earth, surveying and making maps of large areas. It is used to establish and monitor control points, being the basis for all survey work.

2.4. Development of Surveying

During the last two decades, surveying science has witnessed tremendous progress in all of its procedures regarding both field and office techniques. Surveying is no longer about just measuring distances and angles, but rather, has gone beyond many other applications, such as GNSS, laser scanning, digital photogrammetry, mobile mapping, Geographic Information Systems (GIS), and many other surveying techniques and technologies, all of which are defined with the term known as Geomatics.

On the same subject, an organization in the International Surveying (FIG) issued No. (2) in 1991, defined the surveying and surveying profession as follows:

A surveyor is a professional person with the academic qualifications and technical expertise to conduct one, or more, of the following activities;

- to determine, measure, and represent land, three-dimensional objects, point-fields, and trajectories;
- to assemble and interpret land and geographically related information.
- to use that information for the planning and efficient administration of the land, the sea, and any structures thereon; and,
- to conduct research into the above practices and to develop them.

The detailed tasks of surveyor's professionals may involve one or more of the following activities which may occur either on, above, or below the surface of the land or the sea and may be carried out in association with other professionals:

1. The determination of the size and shape of the earth and the measurement of all data needed to define the size, position, shape, and contour of any part of the earth and monitor any change therein.
2. The positioning of objects in space and time as well as the positioning and monitoring of physical features, structures, and engineering works on, above, or below the surface of the earth.
3. The development, testing, and calibration of sensors, instruments, and systems for the above-mentioned purposes and other surveying purposes.
4. The acquisition and use of spatial information from close range, aerial, and satellite imagery and the automation of these processes.
5. The determination of the position of the boundaries of public or private land, including national and international boundaries, and the registration of those lands with the appropriate authorities.
6. The design, establishment, and administration of Geographic Information Systems (GIS) and the collection, storage, analysis, management, display, and dissemination of data.
7. The analysis, interpretation, and integration of spatial objects and phenomena in GIS, including the visualization and communication of such data in maps, models, and mobile digital devices.
8. The study of the natural and social environment, the measurement of land and marine resources, and the use of such data in the planning of development in urban, rural, and regional areas.
9. The planning, development, and redevelopment of property, whether urban or rural and whether land or buildings.

10. The assessment of value and the management of property, whether urban or rural and whether land or buildings.
11. The planning, measurement, and management of construction works, including the estimation of costs.

When carrying out the above-mentioned activities, the surveyors take into consideration the legal, economic, environmental, and social aspects that are relevant and influential in each project.

2.5. Surveying Principles

Surveying, like other sciences, is based on strong engineering principles, as follows:

- Working from the whole to the part: This is one of the most important principles on which survey works are based, as it prevents the accumulation of errors included in every surveying method and facilitates the process of controlling them. For example, looking at the actual practice of surveying work, one may observe that surveying processes start from control points at the national, regional, and then local levels.
- Economy and Accuracy: Surveying works should be carefully planned and match the required accuracy by their application accuracy, with the least possible costs, given that high accuracy means more time and effort, and therefore more cost.
- Repetition of Observations: Professional surveying practice requires taking additional measurements and observations to ensure the accuracy of the work, and examination of the results from different locations and aspects. Repetition of observations helps to find an accurate value of the observed feature.
- Relativity: Determining a specific location based on the known position of another feature is called relative location. For example, relativity is clear in Real Time Kinematic (RTK) GNSS survey applications, where observations of features to be surveyed are achieved, using a fixed reference receiver and a moving one whose position is computed relative to the fixed point; modern Continuously Operating Reference Stations (CORS) networks such as the Kingdom of Saudi Arabia (KSA) CORS (KSA-CORS are also used to serve this requirement regarding single receiver GNSS-RTK techniques. Absolute location identification without the employment of a reference fixed point requires accurate devices, advanced processing, precise tuning, and a large amount of monitoring time. Therefore, most of the survey work is carried out relative to a precise fixed reference.

2.6. Surveying and Error Management

2.6.1. Error Theory

Due to the nature of surveying, errors are inherited into every surveying process with their size and type varying according to the different surveying methods. Errors included in surveying processes can be removed or avoided based on several factors varying from the surveyor's experience to the accurate handling of instrument setup procedures, correctly calibrating the tools and devices used, etc. Surveyors should be aware and have good knowledge of the error sources so as to be able to avoid them and, if occurred, detect them in time. The surveyor should also be aware of the degree of accuracy required in the measurements and the exact scope of the surveying process. This knowledge will guide him in choosing the most suitable survey method, as well as the quality of the necessary equipment and the experience of those working on it.

On the other hand, the various surveying works involved in road design, construction, and maintenance, should not be carried out with the same degree of accuracy. Rather, one has to give each work a specific amount of accuracy in relation to the importance and scope of this work. For example, surveys regarding road feasibility studies require a different level of accuracy than road final design.

2.6.2. Concept of Measurement and Error

Most of the survey work carried out by the surveyor includes a process of measurement of distances, angles, directions, etc. These measurements require several successive steps to be achieved; for example, regarding taking TPS observations, such steps include focus, orientation, matching, and reading, getting at the end an individual value of the observed point's location.

Measurements are generally subject to change, variance, and error as a result of the imperfection found in human senses, especially sight, hearing, and touch, the impossibility of making perfect measuring devices and tools, as well as because of the continuous change in environmental conditions such as temperature, air pressure, humidity, wind, Earth's gravity, etc. For example, if one wants to measure a distance between two points with a measuring tape several times in a row, and there is a change in temperature during the measurement process, this will cause a change in the length of the measuring tape, resulting in varying tape readings. As a result, if this alteration in measurement conditions is taken into account, the obtained distance will be incorrect.

It is clear from the above that all measurements are subject to change, and there is never a completely correct measurement. The phenomenon of change in measurements is a natural thing that should be expected, even if the measuring conditions are fixed and do not change. This change in measurements is known as the true error (*K*), which is the difference between the measured value and the true value of the factor.

$$\boldsymbol{K = X - H},$$ Equation 2-1

where:

K = error in measurement.

X = measured value.

H = correct value.

2.6.3. Error Sources

All measurements, no matter how accurate they are, are always accompanied by an error, due to several factors, the most important of which are as follows:

- Personal Errors: Errors occurring from the surveyor himself, due to imperfections in his senses such as hearing, sight, and touch. Examples of personal errors are, wrong monitoring of the target, error in estimating gradations, error in recording information, etc. Such errors are usually caused due to indifference and negligence during the execution of the work.
- Instrumental Errors: Errors occur due to the imperfection of surveying devices and tools. Given that surveying instruments consist of several parts produced by different materials with several degrees of stability, hardness, and resistance to external influences, measurement errors may occur. Despite the development of modern surveying equipment, it cannot reach an absolute degree of perfection and accuracy due to the difficulty of making a fixed device that is not affected by natural factors.
- Natural Errors: Errors due to the continuous change in environmental factors such as wind, heat, humidity, refraction of sunlight, etc. It is worth noting here that, in the hot climate of the Kingdom, attention should be paid to these changes that cause measurement errors. As result of these conditions, it is advised not to measure at too high a temperature. Surveying instrument's expected errors and operation conditions posed by the instrument's constructor should be always taken into account. Instruments like TPS should not be setup without an umbrella that protects it from the heat of the sun, which may cause the expansion of some of its parts and thus give incorrect readings.

2.6.4. Types of Errors

Measurement errors can be classified into the three types described in the following sections.

2.6.4.1. Blunders

Blunder errors are due to the following factors:

- Observer's lack of interest or as a result of his neglect or ignorance or forgetfulness.
- Inaccuracy of recording information.
- Defects in the measurement tools themselves.

Examples of such errors include; confusion between numbers and letters similar to each other, omissions in recording a stage of the measurement or the measurement itself, wrong shooting at the target, wrong placement of the pole carrying the target/GNSS receiver, wrong recording of the value, etc. Blunders are considered errors that can easily be detected because of their noticeable impact on the measurement results. To avoid mistakes, surveyors should be focused on work and show extreme care; the following measures should be also taken during surveying work:

- Validation of guidance on the desired target.
- Taking additional observations and making sure of their compatibility.
- Reading the measurement two or more times and making sure that it is written down correctly.
- Calibrating instruments and checking their accuracy before work.
- Designing a sound work method that enables easy detection of errors.

2.6.4.2. Systematic errors

They are the errors that have the character of regularity and usually follow a physical law that can be expressed by a mathematical equation. Given previous knowledge regarding the causes of systematic errors, these can be calculated and deducted from the measurements.

Examples of systematic errors include measuring with a tape at a temperature higher or lower than the temperature at which the tape was calibrated, bending of the tape (tape deflection), inclination, incorrect orientation, incorrect instrument, or target height. Systematic errors can be either removed or avoided:

- Removing systematic errors involves finding the mathematical relationship between these errors and the measured quantity, calculating the amount of correction, and adding it to the measurement. Examples include errors resulting from a change in temperature and pressure in electronic measuring devices, which can be corrected using the pressure-temperature-correction graph found on the respective device.
- Avoiding systematic errors is summarized in carefulness in choosing appropriate measurement methods and making observations in a way that enables avoiding them. Such examples include the error resulting from the inaccuracy of the line of sight while taking the readings in the leveling lines affects the acquired height differences (collimation). This can be avoided by placing the leveling device midway between the front and back points.

2.6.4.3. Random errors

After removing blunders from measurements and correcting systematic errors, there remain small errors known as random errors. Random errors are accidental errors that cannot be easily recognized and identified, because, they do not have a systematic behavior.

These errors include failure to correctly setup the instrument above the station, inclination/failure to adjust the bubble of the pole carrying the target/GNSS receiver, weather fluctuations during the monitoring process, etc. While random error causes are usually unknown, its distribution is subject to the normal distribution: if the frequency of all random errors is recorded and put in a graph, this takes the form of a curve known as the normal error distribution curve (Figure 2-1).

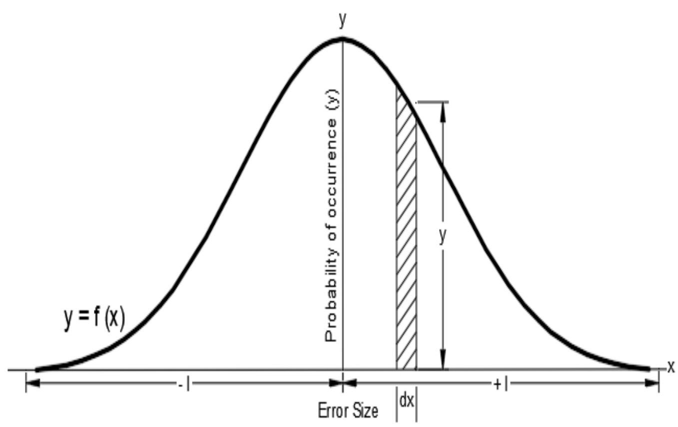


Figure 2-1 Normal Distribution Curve

This curve reveals the following properties regarding errors:

- The probability of an error being negative is equal to the probability of it being positive.
- The probability of a small error is greater than the probability of a large error.
- Big random errors are rare.

The height of its y-axis indicates the accuracy of the measurements, while its flatness means the inaccuracy in the measurements. Although random errors cannot be completely avoided due to the lack of knowledge of their values and algebraic signs, an attempt should be made to reduce the impact of these errors. This is done in one of two ways:

- Choosing accurate devices with a high level of accuracy.
- Measuring the observations several times and taking the arithmetic mean for them.

2.6.5. Accuracy and Precision

Precision and accuracy are the most important and essential elements when measurements take place. Accuracy represents the degree of conformity with a standard, whereas precision relates to the degree of refinement in the performance of an operation or the statement of a result.

Precision measures random errors, i.e., how closely measurements are grouped. The precision of measurement says nothing about whether the measurements are grouped with the correct value.

Accuracy measures how close measurements are to the "correct" value, and is a stronger statement than precision, as it includes both random and systematic errors. To assess accuracy the true result should already be known.

It should be noted here that TPS in their fact sheet usually provide information about precision (repeatability) standards, while GNSS instruments provide accuracy information.

2.7. Map Accuracy Standards

The final product of each survey work is a map of a given scale dictating the area of the actual world that can be presented on the map produced. Based on the requirements of the project and the area mapped, different routes regarding the decision of the scale used can be taken. Map scale controls the horizontal and vertical accuracy of the features displayed in the map, due to the following specification:

- Horizontal Accuracy: The locations of the horizontal features observed shall be accurate and within a Root Mean Square Error ($RMSE_{xy}$) of 0.25 mm on the scale of the map, i.e., less than 0.25 m on a map of 1:1,000 scale (MoMRAH, 2005). Any horizontal feature shall not deviate from its true positions by more than 0.43 mm for a Confidence Level (CL) of 95 %. Resulting accuracies for multiple scales are presented in Table 2-1.
- Vertical Accuracy: Vertical accuracy on a map can be expressed only in the case where contour lines are displayed on the map. Vertical accuracy expressed in terms of RMSE shall be better than one-third (1/3) of the contour interval and the error at any of these heights at CL 95 % shall not be greater than two-thirds (2/3) of the contour interval. This can be verified by taking a selected sample of points on the contour line and

measuring it accurately in nature from the nearest control point. Any contour line is considered acceptable if it falls within the permissible vertical accuracy when moving it in any direction, with no more than 0.5 mm or one tenth (1/10) of the horizontal distance between the contours, whichever is greater in the scale of the map (MoMRAH, 2005). In the case of spot elevation points, RMSE of all elevation points should be accurate to a quarter (1/4) of the contour interval. The error in the elevation points at CL 95 % should not exceed half the contour interval. Contour interval selection varies according to scale, ground type, etc. Typical contour intervals and the corresponding accuracies according to the previous discussion are given in Table 2-2. Vertical accuracies of Table 2-2 regarding contour data are also valid for elevation data resulting from digital elevation data of any form.

Table 2-1 Map Scales and Horizontal Accuracies

Map Scale	**Horizontal Accuracy (m)**	
	RMSE	**Max Error (CL 95 %)**
1:500	0.13	0.22
1:1,000	0.25	0.43
1:2,000	0.50	0.87
1:5,000	1.25	2.17
1:10,000	2.50	4.33
1:20,000	5.00	8.66

Table 2-2 Map Scales, Contour Intervals, and Vertical Accuracies

Map Scale	**Typical Contour Interval (m)**	**Contour Vertical Accuracy**		**Spot Elevation Vertical Accuracy**	
		RMSE	**Max Error (CL 95 %)**	**RMSE**	**Max Error (CL 95 %)**
1:500	0.20	0.07	0.13	0.05	0.10
1:1,000	0.40	0.13	0.27	0.10	0.20
1:2,000	1.00	0.33	0.67	0.25	0.50
1:5,000	2.00	0.67	1.33	0.50	1.00
1:10,000	4.00	1.33	2.67	1.00	2.00
1:20,000	10.00	3.33	6.67	2.50	5.00

2.8. Survey Datum in the KSA

Most of the surveying processes examined in this volume require the cooperation of multiple organizations and authorities. This situation establishes the necessity for the employment of common and accurate horizontal and vertical datum along with consistent and precise control survey procedures to provide secure accurate location of fixed works and rights of way.

A datum provides a reference frame for surveying and mapping positions and heights on the surface of the Earth. Generally, a datum is used as a basis, for computing derivative horizontal and vertical positions on the surface of the Earth. A set of control points (geodetic reference points forming a geodetic network) of known positions are used for the implementation of this datum.

The national geodetic datum is the primary reference for all surveying, planning, and project implementation works nation-wide, and its presence is a basic requirement for any country in the world. It is the cornerstone of every map production project and a strong and accurate link to all road studies, engineering plans, and cadastral data at the national level.

The geodetic network is a set of fixed and monumented points whose location (coordinates) are precisely known in relation to the surface of the earth and is considered as the general frame to which horizontal and vertical data are referenced when making topographic maps. The existence of this network is necessary since all survey works from road design to cadastral mapping, refer to the same level of comparison. Thus, correct information can be extracted when comparing the survey data generated in any process (such as a real estate survey) with the cadastral data resulting from another process, such as utilities maps.

A national geodetic network can take different forms. Historically, geodetic networks are established using the triangulation method, that is, measuring angles, supported by astronomic observations, and baseline lengths to determine the coordinates of all network points. Nowadays, national geodetic networks are supported by GNSS observations making it easy to support and update them.

Specifically, throughout the territory of the Kingdom of Saudi Arabia, the Saudi Arabia National Spatial Reference System (SANSRS) is applied. SANSRS is a consistent reference system that defines Cartesian coordinates, longitude, latitude, height, scale, gravity, and orientation. SANSRS precisely determines locations in the territory of the Kingdom of Saudi Arabia, as well as quantifies changes in the Earth’s surface and its gravity field in space and time.

The SANSRS consists of the following geodetic components (GASGI, 2022) (Figure 2-2):

- National Geodetic Reference Frame (KSA-GRF).
- National Vertical Reference Frame (KSA-VRF).
- Geoid Model (KSA-GEOID).

Among them, KSA-GRF17 realizes the basic Horizontal Datum used in the KSA, while KSA-VRF14 and KSA-GEOID21 realize the respective Vertical Datum. The SANSRS was implemented with the cooperation of the Geodetic Reference Focus Group members, namely (GASGI, 2022):

- Ministry of Defense - General Directorate of Military Survey (GDMS).
- Ministry of Municipal, Rural Affairs and Housing (MoMRAH).
- King Abdulaziz City for Science and Technology (KACST).
- General Commission for Survey (GCS).
- Saudi ARAMCO.

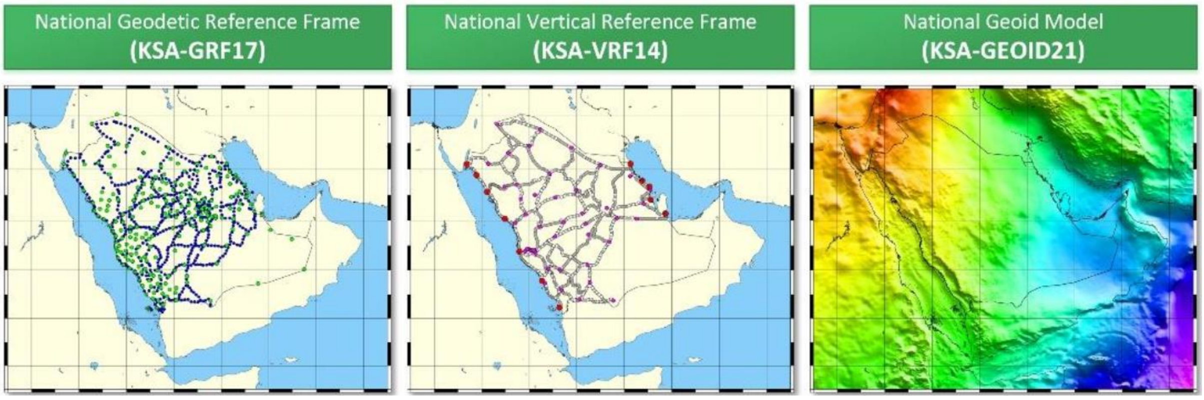


Figure 2-2 Saudi Arabia National Spatial Reference System (GASGI, 2022)

Council of Minster's decision No. 90 dated (5/2/1442 H) (GASGI, 2022) mandates the General Authority for Survey and Geospatial Information (GASGI) to adopt, implement and utilize the SANSRS in its activity, and to promote the SANSRS use and manage its implementation in the KSA.

The SANSRS has to become the foundation for all national geospatial products and is to be used for mapping and charting in a wide range of scientific and engineering applications. All surveys in the Kingdom are mandated to be linked to the SANSRS. As a result, all mapping, planning, design, and construction for every new transportation project or any improvement of existing transportation projects within the Kingdom of Saudi Arabia should be based on SANSRS (GASGI, 2022).

2.8.1. Horizontal Datum

A horizontal datum is a reference frame for precisely representing the position of locations on the Earth's surface in latitude and longitude or another coordinate system. Horizontal datums are implemented with brass monuments inserted into the Earth's surface while their precise positions are determined using triangulation and trilateration techniques.

2.8.1.1. SANSRS definition

National Geodetic Reference Frame (GRF), known as KSA-GRF17 is the latest realization of the National Geodetic Reference System (GRS) developed in 2019. The KSA-GRF17 is based on the International Terrestrial Reference Frame 2014 (ITRF2014) at the epoch of 2017.0, which is the epoch that both reference frames coincide. At that epoch, the coordinates are the same in both reference frames, but they diverge with respect to each other at any epoch different from 2017.0 due to the Arabian plate rotation (GASGI, 2022).

Formally, Kingdom of Saudi Arabia Geodetic Reference Frame 2017 (KSA-GRF17), the unified geodetic reference frame for the Kingdom of Saudi Arabia (KSA-GRF), is defined in such a way that:

- it coincides with ITRF2014 at epoch 2017.0, and,
- is co-moving with the stable part of the Arabian tectonic plate.

Following this definition, the general transformations linking KSA-GRF17 to ITRF2014, for both station positions and velocities, is given relatively to the three signed components of the Arabian Plate rotation pole vector (or angular velocity) estimated for KSA-GRF17 (GASGI, 2022).

Fortunately, SANSRS is accompanied with the KSA-CORS which not only serves for the establishment, definition, deployment, and maintenance of the KSA-GRF, rather than, it provides the user community with access to the KSA-GRF by means of data distribution both for postprocessing and in real time. Specifically, the following positioning services are provided (GASGI, 2022):

- Access to Receiver Independent Exchange Format (RINEX) files from the reference stations.
- Real-time Network Differential GNSS (Network DGNSS) at an accuracy level of decimeter at any point within the Kingdom.
- Network RTK at an accuracy level of centimeter in selected areas.
- Real-time single station RTK at centimeter accuracy near KSA-CORS stations (depending on the distance to KSA-CORS stations).

2.8.1.2. Projection

The Universal Transverse Mercator (UTM) projection, on several zones, is used to project SANSRS geodetic and geocentric coordinates and assign cartographic coordinates on the earth's surface throughout the KSA. Specifically, zones 36N, 37N, 38N, 39N, and 40N are adopted, depending on the location of the project under consideration.

Moreover, in order to make SANSRS available in the different geodetic and GIS software, GCS initiated the procedure of registration of all SANSRS components in both European Petroleum Survey Group (EPSG) registry, and ISO geodetic registry, assigning the values of the following Table 2-3 and

Table 2-4.

Table 2-3 SANSRS Components Registered by GASGI in EPSG Database (GASGI, 2022)

#	NAME	CODE	TYPE	Remarks
1	Kingdom of Saudi Arabia Geodetic Reference Frame 2017	1268	GeodeticDatum	Saudi Arabia – onshore and offshore
2	KSA-GRF17	9331	GeodeticCRS (geocentric)	
3	KSA-GRF17	9332	GeodeticCRS (geographic 3D)	

#	NAME	CODE	TYPE	Remarks
4	KSA-GRF17	9333	GeodeticCRS (geographic 2D)	
5	KSA-GRF17 / UTM zone 36N	9356	ProjectedCRS	
6	KSA-GRF17 / UTM zone 37N	9357	ProjectedCRS	
7	KSA-GRF17 / UTM zone 38N	9358	ProjectedCRS	
8	KSA-GRF17 / UTM zone 39N	9359	ProjectedCRS	
9	KSA-GRF17 / UTM zone 40N	9360	ProjectedCRS	
10	Ain el Abd to KSA-GRF17 (1)	9362	Coordinate Transformation	Initial 3-parameters transformation, accuracy: 2 m
11	Ain el Abd to KSA-GRF17 (2)	9363	Coordinate Transformation	Transformation grid (IGN format), accuracy: 5 - 10 cm
12	ITRF2014 to KSA-GRF17 (1)	9334	Coordinate Transformation	15-parameters transformation, accuracy: 1 mm
13	MTRF-2000 to KSA-GRF17 (1)	9361	Coordinate Transformation	7 parameters transformation, accuracy: 5 -10 cm
14	KSA-GRF17 to WGS 84 (1)	9383	Coordinate Transformation	Derived from ITRF2014 to KSA-GRF17. Valid at 2017.00 and degrading by approximately 2.5 cm per year
15	Kingdom of Saudi Arabia Vertical Reference Frame Jeddah 2014	1269	VerticalDatum	Saudi Arabia – onshore
16	KSA-VRF14 height	9335	VerticalCRS	Orthometric heights
17	KSA-GRF17 to KSA-VRF14 height (1)	9355	Coordinate Transformation	Geoid model (GRAVSOFT format); accuracy: 2 - 20 cm

Table 2-4 SANSRS Components Registered by GASGI in ISO Geodetic Registry (GASGI, 2022)

#	NAME	CODE	TYPE
1	KSA-GRF17 - XYZ	777	Geodetic CRS
2	KSA-GRF17 - LatLonEHt	778	Geodetic CRS
3	KSA-GRF17 - LatLon	779	Geodetic CRS
4	KSA-GRF17 / UTM zone 36N	955	Projected CRS
5	KSA-GRF17 / UTM zone 37N	956	Projected CRS
6	KSA-GRF17 / UTM zone 38N	957	Projected CRS
7	KSA-GRF17 / UTM zone 39N	958	Projected CRS
8	KSA-GRF17 / UTM zone 40N	959	Projected CRS
9	KSA-VRF14 - OHt	780	Vertical CRS
10	ITRF2014 to KSA-GRF17 [GASGI v1]	781	Transformation
11	KSA-GRF17 to KSA-VRF14 [GASGI v1]	782	Transformation

2.8.1.3. Coordinate transformations

The well-known 7-parameters (Helmert) transformation is used to transform geodetic coordinates between older International Terrestrial Reference System (ITRS) based reference frames and KSA-GRF17. Helmert transformation, also known as "Bursa-Wolf" formula and it involves three parameters for translation, three for rotation, and one for scaling between the two involved reference frames. The values of the parameters for some of the transformations can be provided by the respective coordinate transformations registered in the EPSG, and from SANSRS Implementation Guidelines (GASGI, 2022).

Transformations between KSA-GRF17 and Ain Al Abd datum are performed with the aim of a transformation grid representing several values of the translation parameters of the respective Helmert Transformation. The grid is dense enough to provide accuracies in the order of 10 to 100 mm. The interested reader is cited to SANSRS Implementation Guidelines (GASGI, 2022) for further information about the respective transformation methodology.

Furthermore, a GRF transformation tool is provided by GASGI that assists the process of transforming coordinates between the above-mentioned GRF.

2.8.2. Vertical Datum

The Vertical datum is used to determine elevations of points on the surface of the Earth or depths underwater. SANSRS includes KSA-VRF14 (Jeddah 2014) which is the latest realization of National Vertical Reference System (NVRS) in the Kingdom of Saudi Arabia. KSA-VRF14 was defined based on satellite altimetry, Gravity field, and steady-state Ocean Circulation Explorer (GOCE) data, as well as in situ observations from tide gauges, terrestrial national gravity, and leveling networks. (GASGI, 2022). KSA-VRF14 primary benchmark is the JEDDAH Tide Gauge Benchmark (TGBM) TGBM-8, and is implemented by two physical realization techniques, as follows:

Physical Realization Benchmarks:

During SANSRS development the existing Geodetic Leveling Network (GLN) was re-established and densified in Second Order / Class I accuracies. New and replaced benchmarks were installed, measured, and computed throughout the KSA. As such, the regeneration of the GLN was to bring it to its original point density and quality, suitable to serve as the basis for future development within the Kingdom. GLN consists of 3893 physical benchmarks over 88 leveling lines spanning 22,869 km of leveling network (GASGI, 2022).

Physical Realization Grid Interpolation (KSA-GEOID21):

GASGI has developed an accurate, geocentric geoid model KSA-GEOID21, combining all available relevant data in the Kingdom, such as land gravity data, ship-borne gravity points, etc. The computed geoid model allows to reveal the gravimetric KSA Geoid on a grid with a resolution of 2 x 2 km, finally providing altimetric accuracy of about 20 mm at any point within the Kingdom (GASGI, 2022).

Furthermore, a KSA-GEOID21 Height Computation Tools is provided by GASGI for Geoid height computation and the creation of geoid interpolation grids.

All elevations involved in road projects should be based on the KSA-VRF14. If an alternate datum is used then it should be clearly stated, alongside with the correction applied to adjust the elevations to the KSA-VRF14.

2.8.3. Units

The unit of linear measurement in SANSRS is the meter and the unit of angular measurement regarding geodetic coordinates is decimal degrees. Table 2-5 summarizes measurement units used for the SANSRS.

Table 2-5 Measurement Units

Measurement Framework	Unit of Measurement	Measurement System Properties
Spherical: Geographic Coordinate System Longitude and Latitude	**Angular:** Decimal Degrees	**Ellipsoid:** World Geodetic System 1984 (WGS 84) **Horizontal Datum:** KSA-GRF **Vertical Datum:** KSA-VRF
Projected: UTM Zones 36N, 37N, 38N, 39N, and 40N Easting and Northing	**Linear:** Meters	

2.8.4. Other Systems

Two main types of geodetic frames were used in the Kingdom of Saudi Arabia before the establishment of KSA-GRF (GASGI, 2022):

- Geodetic frames were based on the National Geodetic Network (NGN) established in the 1960s; the resulting frame was called Ain Al Abd datum (AAA). Its Reference ellipsoid is Hayford 1909 (International, 1924). It was mainly used by ARAMCO before the introduction of GPS/GNSS. NGN no longer meets the requirements of accuracy, speed, and effectiveness necessary for survey work at the national level.
- With the introduction of ITRS after the establishment of GNSS techniques, the geodetic frames used in the KSA were several national realizations of ITRS via global realizations called International Terrestrial Reference Frames (ITRF) which added the last year of used data. Table 2-6 summarizes such ITRS – based reference systems. An example of this system is the Saudi Geodetic Network (SGN) established by MoMRAH.

Table 2-6 ITRS Based Reference Frames Used by GR-FG Members, (GASGI, 2022)

	ARAMCO	GDMS-GCS	MoMRAH
ITRF-based systems	WGS84 based on ITRF94 at epoch 1998.0	GCS-CORS ref based on ITRF2000 at epoch 2003.1998	Saudi Geodetic Network (SGN2000) MTRF-2000 based on ITRF2000 at epoch 2004.0

It must be pointed out that these systems are here reported for the sake of completeness. It is necessary, from now on, that all entities undertaking survey work for road projects throughout the Kingdom to base their work on the implementation of the SANSRS.

Transformation methods for any other geodetic system used in the past in the Kingdom, such as local systems, should be provided by the respective authorized organizations. Local systems, can be also referenced in KSA-GRF by estimating a 2D polynomial transformation, e.g., affine or 2nd order polynomial transformation. This can be easily done combining at least three local system control points coordinates, to their coordinates measured using GNSS methods and KSA-CORS.

2.8.5. Geodetic Reference Points

Geodetic control points are part of large sets of coordinated control points that include GDMS points, aerial control points, and other public or private project control points. Geodetic control points are monumented and marked points with precisely measured horizontal and/or vertical locations. Geodetic control points are used as a basis to determine the positions of alternate coordinated control points. Each surveying authority holds details on the location and coordinates of surveying geodetic reference points within its own jurisdiction. During the realization of SGN, MoMRAH has established 13 control points of the 1st order and 659 of the 2nd order, to help MoMRAH's surveying department to play the important and vital role that it has regarding surveying and maps at the national level over the Kingdom.

The usage of geodetic reference points nowadays tends to be minimized with the introduction of CORS, i.e., a network of reference stations that is continuously operating tracking GNSS satellites and streaming the data to a control center. Then GNSS data are subsequently transmitted to registered users, i.e., surveyors. Under the framework of SANSRS, the KSA-CORS has been developed with 209 active stations providing Network RTK, single base RTK and DGNSS coverage (Figure 2-3).

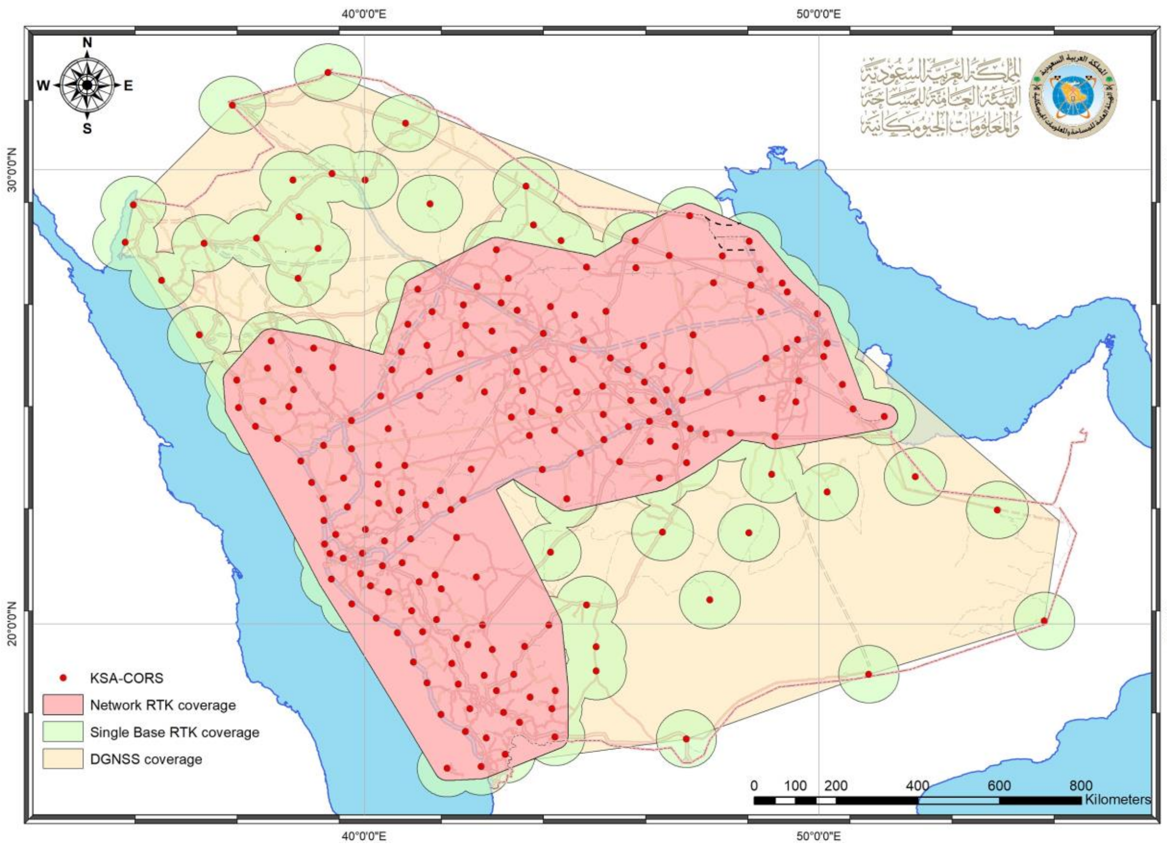


Figure 2-3 KSA-CORS Network Coverage (GASGI, 2020). Up to date information can be provided in https://ksacors.gcs.gov.sa/

The KSA-CORS network aims to provide a reliable and accurate GNSS positioning service across the network. As well, as to realize, distribute and maintain the national geodetic reference frame KSA-GRF17. KSA-CORS provides the users with the following products:

- GNSS Raw Data Download.
- Network RTK, RTK and DGNSS.
- Online Post-Processing Service.
- Geoid height.

Details about KSA-CORS are also presented in Section 10.3 of this Volume.

2.9. Surveying Methodologies

There are many land surveying methodologies related to technologies and appropriate instruments. Choosing which is best is a matter that involves the nature and topography of the area, the size of the project, the required accuracy, and the shape and type of the final product of the survey process. Seven basic surveying methodologies should be used during surveying for road design, construction, and monitoring, as follows:

- TPS Surveying.
- Leveling.
- GNSS Surveying.
- Aerial Surveying.
- Echo-sounding Surveying.
- Terrestrial Laser Scanning.
- Mobile Mapping Systems.

It should be noted that these methodologies can be combined so as to improve the speed of completion, minimize the relative cost and increase the degree of accuracy, as well as the final product's quality. In the next paragraphs, a brief description of each method will be given.

There are several other minor-auxiliary methodologies employed in surveying works. For example, TPS and GNSS terrestrial methods are supported by measuring distances which is the simplest and least expensive method of surveying, as it is based on simple measuring tools (tapes, poles, Electronic Distance Measurement (EDM)). In this way, points and features are identified by measuring their distance from fixed reference points. Distance measuring should be used to measure only horizontal distances; therefore, their employment is limited to rather flat sites.

2.9.1. TPS Surveying

Total Station, Total Point Station, or TPS surveying is the oldest surveying method, and one of the most widely used, due to their ease and simplicity of use, in addition to their ability to provide high accuracy angle and distance measurements, resulting in very accurate relative positions.

Modern Total Stations are devices that combine classical theodolites with distance measurement and recording units; as such Total Stations measure with high accuracy horizontal and vertical angles, and distances and record the results in their internal memory or even attached controllers with appropriate management software. While vertical angles and distance measurements can be combined to provide height differences, these are not as accurate as the leveling devices examined in the next section.

With the introduction of recording units, these devices have eliminated older analogical methods where observations were read from the device and written on survey notes, possibly the largest source of blunder errors in surveying in the past. Significant improvements have also occurred in TPS surveying in recent years, with the introduction of robotic TPS that allow remote operation. They have also been integrated with GNSS devices, providing the ability to determine their position with appropriate GNSS methods such as Fast Static, RTK, and Post Processing Kinematic (PPK) examined in Chapters 10, 11, and 12 of this Volume.

With the introduction of modern kinematic GNSS techniques such as RTK and PPK, the significance of TPS surveying has been degraded. Nevertheless, TPS surveying should be selected in several cases, especially when dealing with small-scale and high-accuracy control surveys, in urban and forested (high-vegetated) environments where kinematic GNSS techniques efficiency degrades, as well as, where it is dictated by the accuracy requirements (e.g., construction).

2.9.2. Leveling

The scope of leveling is to measure or verify the height of specified points relative to a datum. Leveling is performed with the use of levels. Levels are optical instruments used to establish or verify points in the same horizontal plane and are commonly used in surveying to measure elevations. The process of leveling, also known as direct or differential leveling, measures vertical distances directly on a graduated rod with the use of a leveling instrument. Leveling regarding road projects is mainly employed in control surveys and construction.

2.9.3. GNSS Surveying

Global Navigation Satellite System (GNSS) refers to several constellations of satellites broadcasting signals from space that transmit positioning and timing data to GNSS receivers which then use this data to determine their location; GNSS provides global coverage. GNSS has been introduced as a term that generalizes the notion of a Global Positioning System (GPS), which was the first actual operating satellite constellation serving this purpose. GNSS includes Europe's Galileo, the USA's NAVSTAR GPS, Russia's Global'naya Navigatsionnaya Sputnikovaya Sistema (GLONASS), and China's BeiDou Navigation Satellite System.

In order to define a precise horizontal location on earth, signals from at least three satellites are required. For the determination of vertical position at least four satellite signals are needed. Due to its positioning capability, GNSS is widely used for surveying. A single GNSS receiver observing sessions of only a few hours can yield three-dimensional positions with accuracies of a few centimeters. Differential techniques evolved soon after the introduction of the first constellation (GPS), using base stations (Figure 2-4), allow to accurately and, in some cases, instantly, define the position of a point on the surface of the earth.

GNSS survey guidelines continually evolve with advancements in equipment and techniques. Changes to these guidelines are expected as these advancements occur. Recent changes include the adoption of Inertial Measurement Units (IMU)/inclinometers on GNSS receivers allowing thus to determine point positions regardless of the slope of the pole on which the receiver is mounted.

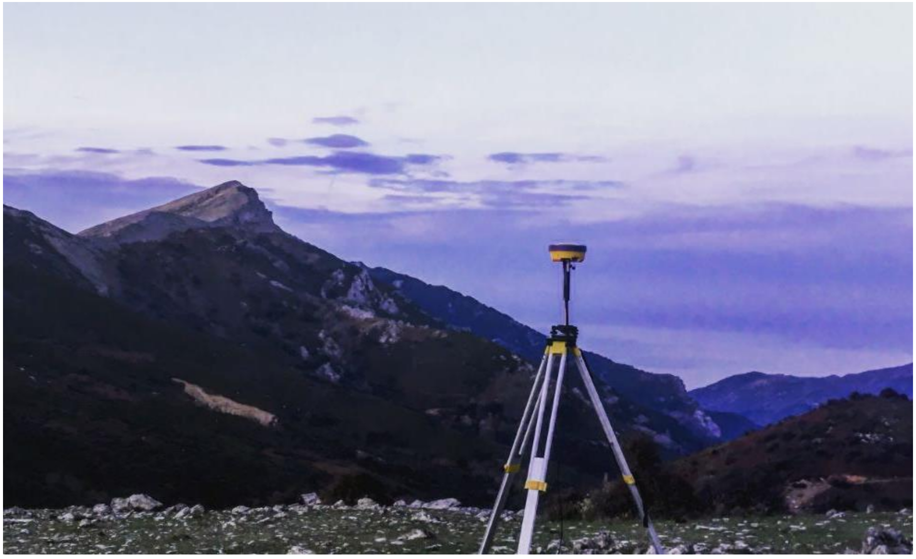


Figure 2-4 GNSS Base Station

There are different types of GNSS surveying techniques such as Static, Fast Static, RTK, PPK, etc. Static and Fast Static GNSS techniques should be employed in large area control surveys, while RTK and PPK should be used in rural and non-forested environments, given that accuracy requirements are met. For details, refer to Chapters 10, 11, and 12 of this Volume.

2.9.4. Aerial Surveying

It is the science concerned with collecting map data utilizing aerial photographs, satellite images, and aerially collected laser data, using a camera and other equipment. Aerial surveying products are not limited to making topographic maps only, since they can be used in many other scientific fields such as geology, earth sciences, soil classification, remote sensing, etc.

Aerial survey methods with the aim of photogrammetry have developed in recent years in a way that opened wide horizons in the production of maps. The introduction of Unmanned Aerial Vehicles (UAV) and relative technologies such as Structure from Motion (SfM) and Multi-View Stereo (MVS) have further boosted their employment, providing the ability to generate accurate colored Point Clouds (Figure 2-5). Moreover, satellite imagery providing coverage of a very large area, with modern sensors providing up 0.3 m Ground Sampling Distance (GSD), can also be used for mapping purposes at appropriate scales. Finally, aerial Light Detection and Ranging (LiDAR) with its unique characteristics can be used to provide accurate Digital Terrain Models (DTM) with automatic vegetation removable, where other photogrammetric or ground survey techniques require multiple resources or even fail.

Aerial surveys are currently considered the most popular surveying method. It has been practically proven that aerial survey methods save at least 50 % of the time, effort, and money needed to produce maps compared to ground methods. They are usually used for cadastral mapping, topographic survey for large areas or digital terrain modeling survey, preliminary/final design of roads, and other projects. Aerial Surveys can be used to provide several products such as vector maps, Digital Surface Models (DSM), orthophotomosaics (proved to be an excellent background for vector information), or even Virtual and Augmented Reality applications.

Figure 2-5 Aerial Survey Generated Point Cloud

2.9.5. Echo-Sounding Surveying

An echo-sounder or Sound Navigation and Ranging (SONAR) is a system determining water depth based on sound reflected to the sea bottom. The system consists of a sound transmitter and a receiver that picks up the reflected echo, electronic timing, and amplification equipment, along with a display for operational control and coverage verification. Echo-sounders may be attached to the hull of the vessel, towed from a surface vessel Figure 2-6. An echo-sounder sends a sound pulse into the water, the sound energy travels through the water to the sea bottom where it is reflected back towards the receiver, where it is timed and recorded.

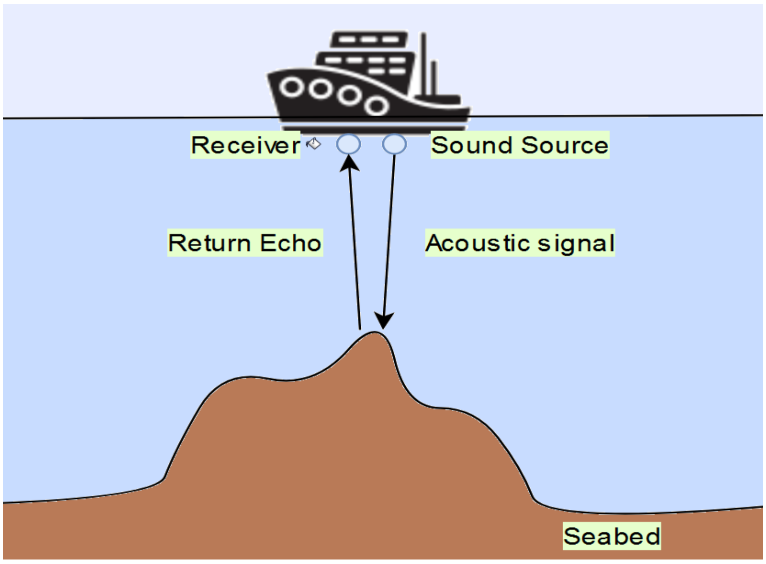


Figure 2-6 Illustration of Echo-Sounding Mechanism

Echo-Sounding is widely used in hydrographic surveying which aims to define the boundaries between physical features of bodies of water such as rivers, lakes, and streams, and the adjacent land areas, as well as the shape of the seabed. Hydrographic surveys primarily focus on phenomena that affect the safety of navigation, publishing that information in a form that enables it to be used for marine navigation.

2.9.6. Terrestrial Laser Scanning

Laser scanning with LiDAR (Vosselman & Maas, 2010), is a method using a laser to measure ranges (distance) by targeting an object and measuring the time light needs to travel towards the object and back. It is an active method, i.e., it provides its own illumination source, hence do not rely on external sources, and can work on night as well as day. Its output is a low noise point cloud, colored by the intensity of the returned signal. LiDAR systems which incorporate calibrated cameras can also return natural color information for each point.

LiDAR sensors static on tripods (terrestrial) are used when a digital 3D representation of an object or earth's surface is needed (Toth & Shan, 2009). Most common applications are in surveying, geodesy, cultural heritage recording, geology, geomorphology, seismology, forestry, atmospheric physics, and laser altimetry. The survey is divided in static surveys from selected stations, in the same sense of a traverse with total station. In fact, their distance measuring methodology is based on the equivalent distance measurement module of total stations. The static LiDAR scanners are called Terrestrial Laser Scanners (TLS). TLS are employed when high-accuracy and high-density point clouds are needed in road-related projects, e.g., side slope and surface surveys, and in as-built surveys.

2.9.7. Mobile Mapping Systems

Mobile Mapping Systems (MMS) are devices that collect geospatial data from a mobile platform, typically fitted with a range of cameras, LiDAR, Radar, echo-sounders, or any number of remote sensing systems, for land, air, or marine applications. The uptake of MMS is fueled by the demand for fast and cost-effective data acquisition of large areas, along with technological developments which address this demand. Two developments are especially important in this context: LiDAR sensors and precise navigation. The common feature of MMS is that they combine all sensors necessary to solve a specific problem, mounted on a common platform. By precise synchronization of all data streams, data collection for a specific application is possible, even in real-time. As such, MMS are typically comprised of two main types of sensors; sensors for positioning, i.e., GNSS, Inertial Navigation System (INS), Distance Measurement Unit (DMI), and sensors for data collection i.e., LiDAR and cameras (Figure 2-7).

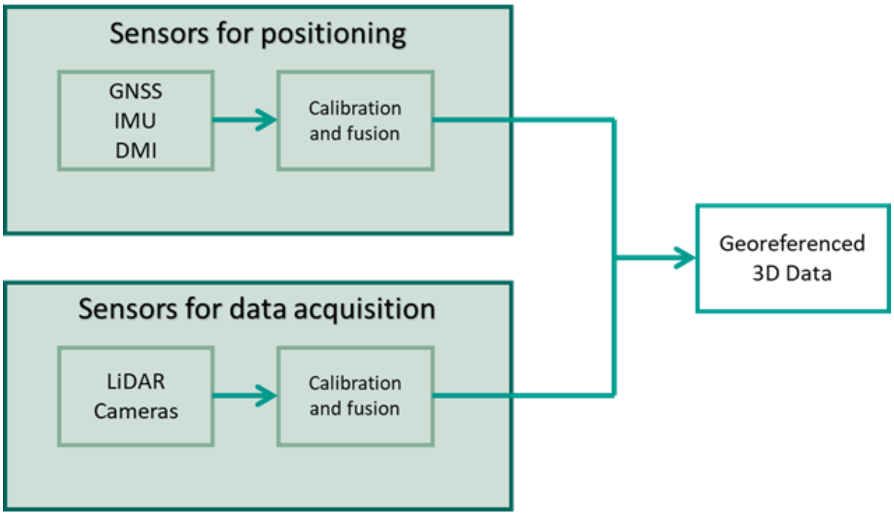


Figure 2-7 MMS Sensors Fusion

The primary output from such systems includes 3D point clouds, georeferenced images and video, GIS data, and digital maps. Hence, MMS are an important source for various applications, including, but not limited to: indoor and outdoor 3D modeling, capturing of GIS data, disaster response high-detail 3D maps, and autonomous vehicles (Elhashash, et al., 2022).

MMS are being used in several land, marine, aerial, and indoor applications including:

- Asset and infrastructure management/maintenance.
- 3D imagery for online mapping tools, street-level views, and mobile apps using map data.
- Road mapping and highway facilities.
- Assessment and monitoring of road surface.
- Rail and utility corridors.
- Pipeline surveys (Bridge, tunnel, etc.).

Given the variety of applications, there is a wide variety of implementations that fall under the generalized MMS category. Selection of the appropriate MMS is depended on the application and its unique characteristics.

3. Survey Requirements for Highway and Road Projects

3.1. Surveying Procedures in Road Projects

During the road project lifecycle, several surveying procedures take place. These procedures being applied to several stages of the lifecycle of road projects are mainly categorized into six categories, namely, Control, Terrain, Topographic, Cadastral, Bathymetric, and Construction, as described in the following paragraphs. Table 3-1 summarizes the applicability of surveying methodologies over surveying procedures for road projects.

Table 3-1 Surveying Methodologies Applicability over Surveying Procedures During Road Projects' Lifecycle

			Surveying Procedures					
			Control	**Topographic**	**Terrain**	**Cadastral**	**Bathymetric**	**Construction**
Surveying Methodologies	**Aerial**	Satellite	-	✓	✓	-	-	-
		Photogrammetry	-	✓	✓	✓	-	-
		LiDAR	-	-	✓	-	✓	-
	Terrestrial	TPS	✓	✓	✓	✓	-	✓
		Level	✓	-	-	-	-	✓
		GNSS	✓	✓	✓	✓	-	✓
		TLS	-	✓	✓	-	-	✓
	Echo-sounding		-	-	-	-	✓	✓
	Mobile Mapping		-	✓	-	✓	-	✓

3.1.1. Control Surveys

The first step regarding every survey project is the establishment of the project's reference with the creation of control points. Project control is performed with the aid of control surveys during which a unified consistent network of physical ground stations (monuments) is to be implemented. These points are the basis for the project's horizontal and vertical reference system. The exact position of each control point is determined and subsequently used for detailed topographic surveys, construction staking, or similar needs. Control surveys ensure that adjacent projects have compatible control and a common reference system. GNSS, TPS, and Levels may be alternatively employed for horizontal and vertical point measurements. Least Squares Adjustment should be used for final coordinate estimation. Control surveys in the KSA shall be all referenced to KSA-GRF17 and KSA- VRF14.

Both horizontal and vertical control surveys and the resulting networks are classified into accuracy orders. The purpose and accuracy order dictate the tolerances allowed. Specifically, regarding road projects:

- First Order classification stands for the highest accuracy and refers to the corridor control regarding road projects. It is more stringent in terms of accuracy requirements compared to the Second Order, which in turn is more demanding than the Third Order.
- Second Order classification for road projects is used to establish project (site) control between tracts bounded by the first-order control network.
- Third Order classification standards establish control for local area projects such as small engineering projects, for local improvements and developments, as well as small topographic mapping projects.
- Lower order control points are constant points used in topographic surveys, e.g., stations.

First, second and third-order control points are permanently marked by monuments established as in Appendix A.

Survey control for the purpose of road surveying, referred hereafter in this code, follow order classification of Table 3-2. Table 3-2 further illustrates the relation between road survey orders and National Oceanic and Atmospheric Administration (NOAA), National Geodetic Survey (NGS) Standard classification (FGCC, 1984), also adopted by GASGI.

Table 3-2 Survey Orders: Road Order vs NGS Standards

Road Survey Order	NGS Standard Order
First	Second order / Class I
Second	Second order / Class II
Third	Third order / Class II
Fourth and Lower orders	Fourth and Lower orders

Control points for road surveys in general are established with the help of GNSS surveys presented in Chapters 10, 11 and 12, especially regarding horizontal control. TPS control surveys (Chapter 7) shall be used for horizontal control in the following cases (CALTRANS, 2016):

- When GNSS methods cannot be applied e.g., mines, tunnels, urban environments.
- When relative precision required by the survey cannot be met by GNSS techniques (e.g., when high accuracy is needed in limited-size sites).

Regarding vertical control, differential leveling shall be used for first, second and third-order accuracy using appropriate survey equipment and practices. Trigonometric heighting and static GNSS methods may also be used in third-order surveys. Heights for fourth-order accuracy purposes shall be provided by differential leveling, trigonometric heighting, or with GNSS methods. Above information is summarized in Table 3-3.

Table 3-3 Suitable Methodologies by Control Type and Order

Survey Order (Road)	Horizontal Control	Vertical Control
First	GNSS[1], TPS[2]	Leveling
Second	GNSS[1,1*], TPS[2]	Leveling
Third	GNSS[1*], TPS[2]	Leveling, TPS, GNSS[1,1*]
Lower	GNSS[3], TPS[4]	GNSS[3], TPS[4], Leveling

NOTES:

1. Static, [1*] Fast Static,
2. Triangulation / Trilateration / Network adjustment
3. Kinematic (RTK, PPK)
4. Traverse

3.1.2. Topographic Surveys

Topographic surveys are used for the collection of field data regarding natural and man-made features in the area of the project under design. Topographic surveys are commonly supported by horizontal and vertical control of third and lower order. Methods and instruments used in topographic surveys depend upon the purpose of the survey, the degree of precision needed, the nature of the terrain to be covered, and the map scale.

Several surveying methods can be used, alternatively or combined, to support topographic surveys. Traditional fieldwork in a topographic survey takes place with TPS by initially creating a basic framework of local horizontal and vertical control points, i.e., stations forming a traverse. Once these points have been established, surveyors locate, measure, and document in field notes, the recognized natural or manmade feature details in the vicinity of each station that will appear on the respective map.

In recent years, wide use of TPS tends to be replaced by GNSS (Chapters 10, 11, 12) methods, i.e., RTK and PPK, since they require a smaller crew, and are not dependent on the rather small line-of-sight vicinity of each station to measure feature details. Nevertheless, TPS methods are still preferred in cases of forested areas with high vegetation, urban environments where GNSS methods are limited by the available sky visibility, as well as when required accuracy dictates so.

Apart from using ground field methods, topographic surveys can be also implemented using aerial photography and photogrammetry (Chapter 13) which is the most efficient method for obtaining large volumes of topographic data. However, photogrammetry is not always able to capture the actual feature details due to vegetation and other location characteristics.

Finally, Mobile Mapping can be used to document existing road surfaces, given the nature of the project and accuracy requirements.

3.1.3. Terrain Surveys

Terrain Surveys aim at obtaining the representation of the surface of the Earth finally constructing a DTM of the area under consideration. DTM is a topographic model of the Earth's surface that can be manipulated by computer programs, referenced to a given coordinate system (datum). Development of DTM enables the aspect of elevation and adds 3D capabilities to design projects; it also assists in procedures such as alignments of roads, location of future buildings, and other types.

It is essential here to distinguish between Digital Terrain Models (DTM), Digital Surface Models (DSM) and Digital Elevation Models (DEM). Although all models describe elevations, the main difference between them is that DEM is a 'bare earth' elevation model which is a superset of DTM and DSM, with DTM being a DEM augmented with features like breaklines and ridges while a DSM being a DTM that includes the natural and human-made features on the earth's surface. The diagram of Figure 3-1 shows the differences between a DTM and DSM., where DTM (cyan line) follows the ground, whereas the DSM (red line) follows the structures on the surface.

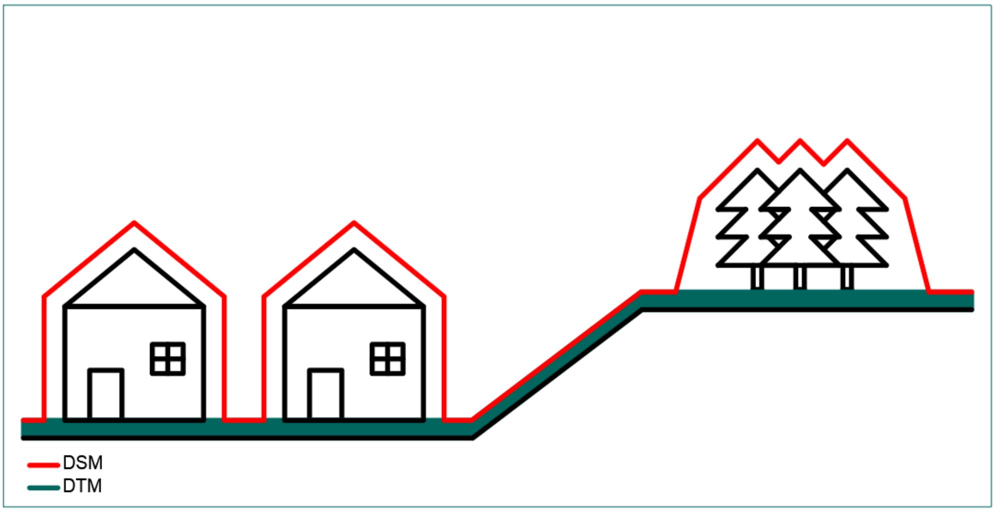


Figure 3-1 Difference between DSM and DTM – DEM

Terrain surveys can be carried out with alternate procedures, or simultaneously with the topographic surveys, depending on the nature of the project. The data provided for the development of a DTM are collected by field survey measurements such as TPS, GNSS, TLS, as well as with aerial methods like Photogrammetry and LiDAR.

3.1.4. Cadastral Surveys

According to (KSA Cabinet Decision No. 54/1437, 2015), landowners in the KSA are entitled to receive compensation for the parts truncated for the planning or the determination and implementation of the main roads' networks. Therefore, the need for cadastral surveys determining public and private properties in the land arises. Cadastral surveying refers to recording the existing landowning status of real properties affected by road projects, based on documents, historical evidence, actual field conditions, and existing standards of practice. It also refers to the initial installment or reinstatement of already established real estate boundaries and monuments. Cadastral Surveys also document the policies, standards, and procedures regarding boundary control of road projects.

Cadastral surveys produce plats, i.e., cadastral maps, drawn to scale, showing the divisions of a piece of land. Plats show the distance and bearing between land section lot corners, sometimes including topographic or vegetation information. Cadastral surveys should provide precise locations of horizontal positions of roads, buildings, public lands, private boundaries, Right-of-Way (ROW) and ROW plats, and other future developments.

Cadastral surveys can be carried out with alternate procedures, or simultaneously with the topographic surveys, depending on the nature of the project. TPS, GNSS, and photogrammetric methods should, alternatively or combined, be employed in cadastral surveys.

3.1.5. Bathymetric Surveys

Bathymetric surveys are the sum of all acquisition technologies and processing methods to create a 3D map of areas covered by water, whether this might be sea, river, or lake. Such surveys are measuring the water column above a large number of points and finally recreate the surface of the seabed and objects lying on it, either shipwrecks or other man-made structures and infrastructures.

Traditionally the primary output of bathymetric surveys are bathymetric charts. Printed bathymetric charts produce contour lines to show depth from a reference surface, usually mean sea level. These contours are known as isobaths. In the digital era, the primary output are DSM files, from which several visualizations can be produced.

Bathymetric surveys for road projects take place near planned bridges and other sites close to and affecting water bodies. The surveying methodologies used are based in echo-sounding (sonar) or aerial LiDAR to accurately measure the depth of the water column (in clear and shallow waters).

3.1.6. Construction Surveys

Prior to the construction activity of any project, the design information has to be transferred from the plan to the ground. This is accomplished in general, by construction surveys and staking. Construction surveys are employed for any construction project, from roads to buildings, tunnels to hydraulic works, and underwater structures. Construction surveys are extremely useful and important since all construction phases are based on their output. Construction surveys in highway construction guide all work from the initial earthwork to the pavement placement, lighting poles establishment, etc. Construction surveys are present even after the end of road construction so as to produce precise as-built drawings.

Standard equipment used in most terrestrial survey operations is also employed for construction surveys such as TPS, GNSS, levels. Construction surveys require final design control network recovery, verification, and thickening. They are typically more demanding in terms of accuracy requirements regarding design phases. Details about construction surveys are given in Chapter 19 of this Volume.

3.2. Road Projects' Lifecycle Phases

In the following sections, general survey requirements regarding all phases of road design, construction, and maintenance are presented. Table 3-4 summarizes and provides tabular information included in the following sections.

3.2.1. Feasibility Study

During the feasibility study/concept design stage, possible directions of road alignments, parking, bridges, and other road-related structures positions are decided. For this purpose, a survey is required to provide the topographic background needed for deciding the general study directions.

Table 3-4 Survey Scales, Instruments, and Types During Road Projects' Lifecycle

			Road Projects' phases				
			Feasibility / Concept Study	**Preliminary Design**	**Detailed Design**	**Construction**	**Operation / Maintenance**
Scale			1:10,000	1:2,000 - 1:5,000	1:1,000 1:500[2] 1:200[3]	N/A	Operation Specific
Surveying Methodologies	**Aerial**	Satellite	✓	-	-	-	-
		Photogrammetry	✓	✓	✓	-	-
		LiDAR	-	✓	✓	-	-
	Terrestrial	TPS	-	✓[4]	✓	✓	✓
		Level	-	✓[4]	✓	✓	✓
		GNSS	-	✓[4]	✓	✓	✓

			Road Projects' phases				
			Feasibility / Concept Study	Preliminary Design	Detailed Design	Construction	Operation / Maintenance
		TLS	-	-	✓	✓	✓
	Echo-sounding		-	-	✓	✓	-
	Mobile Mapping		-	-	✓	✓	✓
Surveying Procedures	Control		✓	✓	✓	✓	✓
	Terrain		✓[1]	✓	✓	✓	✓
	Topographic		-	✓	✓	✓	✓
	Cadastral		-	-	✓	-	-
	Bathymetric		-	-	✓[5]	✓[5]	-
	Construction		-	-	-	✓	✓

NOTES:

1. Only DSM.
2. At intersections.
3. Regarding road improvement.
4. Extensive usages discouraged.
5. Given appropriate circumstances, e.g., bridges.

Feasibility studies should be carried out with the help of orthophotomosaics produced by Aerial Photogrammetric or Satellite imagery, DSM, as well as existing maps. Both aerial and satellite imagery employed should regard new collections. A field reconnaissance survey should also take place to reveal information hidden by vegetation or other obstacles.

Maps produced in feasibility studies and concept design shall be of 1:10,000 scale or smaller. Horizontal and vertical accuracies regarding all map products for feasibility studies are driven by the respective map scale as described in Section 2.7.

Orthophotomaps should be produced by combining orthophotomosaics, DSM generated contours, vector and textual feature data derived from by existing maps, photointerpretation, and field reconnaissance surveys. Only features of considerable size will be included in such maps due to their small scale, e.g., road centerlines, large building, bridges, etc.

The area to be covered by such surveys should follow the details given in Table 3-5, in relation to the initially planned corridor positions.

Table 3-5 Coverage Details About Proposed / Existing Road / Corridor for Feasibility Studies

Project Type	Width of Corridor to be Covered (Imagery, maps, and DTM)
Undefined road corridor	5 km
Defined road corridor	500 m on both sides of the centerline
Road improvement	1.5 km on both sides of the centerline
Existing interchanges	3 x 3 km
Proposed interchanges	5 x 5 km

The deliverables for this stage include as a minimum the following:

- Technical report including processing (calculations) report.
- Project control drawings and tables showing coordinates of control points.
- Digital georeferenced files for the DSM.
- Digital georeferenced files of the final orthophotomosaic.
- Digital orthophotomap including DSM-produced contours, vector, and textual data in editable format.
- Digital orthophotomap sheets in PDF format, ready to be printed to scale.
- Any additional data required by the project owner submitted in digital form.

3.2.2. Road Preliminary Design

Road preliminary design requires a high-quality topographical survey, with the objective of providing a digital terrain model of the roadway corridor, with all the roadway features that exist in the corridor, recorded in three dimensions. As such, the road preliminary design includes a detailed survey of the project site in scales that should be between 1:5,000 and 1:2,000, according to the project's owner decision. Horizontal and vertical accuracies regarding all surveying products for preliminary design are driven by the respective map scale as described in Section 2.7.

Preliminary design surveys should be carried out with the help of manned or unmanned aerial methods, namely, aerial photogrammetry. Satellite imagery shall not be used for road preliminary design. The use of terrestrial methods (TPS/GNSS) is also discouraged in this phase due to their high cost. LiDAR can be used to obtain DTM in cases where actual terrain data cannot be obtained by photogrammetry; in this case, aerial imagery shall also be collected. All data employed for this purpose, aerial imagery, and LiDAR data should regard new collections. Details about photogrammetric and LiDAR surveys are given in Chapter 13 of this Volume.

The area to be covered by these products should follow the details given in Table 3-6 according to the nature of the respective terrain.

Table 3-6 Coverage Details About Proposed / Existing Road / Corridor for Preliminary Studies

Project Type (new roadway or improvement)	Width of Corridor to be Covered (Imagery, maps, and DTM)
Flat terrain (slopes < 10 %)	300 m (150 m on each centerline side)
Hilly terrain (slopes between 10 - 20 %)	400 m (200 m on each centerline side)
Mountainous terrain (slopes > 20 %)	200 m (100 m on each centerline side)

While the extensive use of terrestrial methods in preliminary design is discouraged, first and second-order project control should be established in this phase. Aerial triangulation control points used in the photogrammetric process shall be referenced to this network. GNSS control surveys shall be used for this process, according to the specifications of Chapters 10, 11, and 12. All elevations and coordinates should be related to the KSA-GRF17 and KSA-VRF14 described above.

Spacing between first and second-order control points shall follow Table 3-7 values. Control points shall be established at least at the beginning and the ending of the site.

Table 3-7 Control Point Spacing

Order of survey	Control point spacing
First order (Corridor control)	4 - 10 km
Second order (Project control)	500 - 2,000 m

The survey should produce vector data by locating all existing structures found in the survey width, as well as creating the respective DTM. Data collected will include:

- Property-line boundaries and road-reserve boundaries, palings, fences, and junk yards.
- Buildings, bridges, and culverts including invert of inlet and outlet levels, underpasses, ramps, roads, slip roads, access roads, junctions, and all other surface services and man-made features.
- Wadis, streams, creaks, spot elevations, and other terrain features.

The deliverables for this stage include as a minimum the following:

- Technical report including methods selected and processing (calculations) report.

- Project control monuments, drawings, and tables showing coordinates of control points.
- Digital georeferenced files for the final DTM.
- Digital georeferenced files of the final orthophotomosaic.
- Digital orthophotomap including DTM produced contours, feature vector, and textual data with appropriate symbols and legend.
- Digital orthophotomap sheets in PDF format, ready to be printed in scale.
- Any additional data required by the project owner submitted in digital form.

3.2.3. Final / Detailed Design

Road final/detailed design requires the highest quality topographical survey. As such, survey scales shall be at least 1:1,000 in the main road corridor and 1:500 in intersections. Road improvement projects may require even larger scales e.g., 1:200, especially when improvements are to be taken in the existing road corridor. Horizontal and vertical accuracies regarding all map products in the final design are also driven by the respective map scale as described in Section 2.7.

Final design surveys should be carried out with terrestrial (GNSS/TPS/TLS) or aerial photogrammetry and LiDAR methods of appropriate scale. Satellite imagery does not provide the required accuracies; therefore, it shall not be used in the final design. All data employed for this purpose, aerial imagery, and LiDAR data should regard new collections. Mobile Mapping methods may be also used in road improvement projects given that they provide results of appropriate scale and survey width.

The basic width of the area to be covered shall be at least 3 times greater than the maximum of the planned road corridor width including side slopes, or 120 m, whichever is greater. Maximum planned road corridor width including side slopes can be acquired from the plans of the respective preliminary design. As such, in a corridor of 50 m including side slopes according to the preliminary design, the respective total survey width will result in 150 m, while in a corridor of 30 m the survey width will result in 120 m.

During final design, control, topographic, terrain, and cadastral surveys are required. Control survey monuments, established during preliminary design survey, shall be recovered, and verified during this phase. If terrestrial methods are used, the control network shall be densified to support main surveying activities. All elevations and coordinates should be related to the KSA-GRF17 and KSA-VRF14 described above.

The survey should produce vector data by locating all existing structures within the planned survey width, as well as creating the respective more detailed DTM, with breakline features. Survey data collected in the final design involves all features contained in the preliminary design survey, in higher scale and accuracy with the addition of smaller size features (not visible in the smaller scales of preliminary design), and more detailed terrain features:

- Buildings including gate levels.
- Underground utility services such as manholes, coffer dams, and wells.
- Street light poles, electricity poles, overhead lines, service markers, etc.
- Traffic signs, safety parapets.
- Other small man-made features.

During the final design phase, cadastral surveys shall be also taken place as described in Chapter 18. When data for underwater surfaces need to be collected during the final design, bathymetric surveys with echo-sounding instruments or aerial LiDAR surveys shall also take place as described in Chapter 16 of this Volume.

Deliverables of survey works about final design are identical to the ones described in Section 3.2.2 regarding preliminary design, with the exception of the different scales/methods/details described here.

3.2.4. Construction

In addition to being a surveying procedure, construction is also a phase of road project's lifecycle, during which construction surveys take place. During the phase of construction, as-built surveys are also realized, which aim to check whether the actual construction has been completed as expected according to the design of horizontal alignment, vertical alignment, and cross-sections. As such, in this stage, it is confirmed that the construction complies with the design. As-built survey for road projects includes, but is not limited to, all final built locations of structures, utilities, manholes, valves, storm drains, catch basins, curb, and gutter, pavement, sign structures, light poles, traffic signals, etc. As such, as-built is essential for a detailed topographic survey, utilizing TPS, GNSS, and TLS. Finally, ideal for road corridors' as-built surveys are MMS.

3.2.5. Operation / Maintenance

There are many situations where survey works can be applied during road operation and maintenance, mainly related to monitoring of existing constructions and roadway's surface. These works require the design/construction control network (monuments), which should be properly maintained after road construction is completed. For such purposes, terrestrial instruments are employed such as TPS, GNSS, and TLS.

Another major survey work that may take place during roadway's operation originates from the need for updating its as-built information towards the employment of a BIM. BIM is an Information model that represents structural elements using 3D digital technology while accommodating them with a data source that contains various related information about those elements. Additionally, the 3D modeled elements carry not only geospatial information, but they also contain metadata information that describes those element's attributes. Such surveys should take place with MMS which are ideal for this kind of use. Details about BIM in road construction and Mobile Mapping Surveys are given in Chapters 15 and 20 of this Volume.

3.3. Roles and Responsibilities

In the following paragraphs, the main roles regarding surveys with the responsibilities applied to each particular role are defined.

3.3.1. Client

The client as the owner of the project, poses requirements and standards, thus having the responsibility to ensure that the survey work is completed as specified. Consequently, the client is responsible for appointing the designer/consultant (regarding road design) and contractor (regarding road construction), which should meet the originally set requirements. During designer/consultant and contractor selection the client shall ensure that the personnel/staff selected meets the qualification and experience requirements.

3.3.2. Designer / Consultant

All survey work made by the designer/consultant's surveyor during any stage of the road project design shall meet the accuracy requirements and general guidelines posed by the Client. Designer/consultant shall appoint a surveyor in charge, responsible for all designer/consultant's survey work.

3.3.3. Contractor

The contractor's survey team is responsible for performing the desired construction survey. Allowable accuracy requirements set by the client shall be respected and all surveying/staking work needs to be within these requirements. Contractor team shall appoint a survey in-charge which is responsible for all survey work made by the contractor.

3.3.4. Responsibility for Survey Data Accuracy

During the design of the project, the designer/consultant survey company shall be responsible for any error occurring in the survey work. During the construction stage, the contractor shall be responsible for the accuracy of the survey work throughout and after the construction.

4. General Guidelines for Field Survey Work

4.1. Planning Surveying Works

4.1.1. Field Survey Planning

Before starting the survey work in the field, it is necessary to fully understand the purpose behind it, identify the final result to be achieved, and plan how these results will come out. Any previous survey work regarding the site under consideration should be collected: reports, maps, existing control points, etc. Moreover, existing maps, aerial photos, and satellite imagery regarding the site should be studied in advance. Section 4.1.2 provides such information. Other publicly available mapping services, such as google maps, etc., should be also studied prior to the beginning of the work.

An initial field survey helps also to determine the nature of the site and develop a vision or plan, for how to implement the work in the most efficient way in terms of available resources, trying not to overestimate the accuracy and to harmonize it with realistic costs. The supervising survey engineer, the survey team entrusted with the work, and the beneficiary of the work (whenever possible) should participate in this plan preparation (MoMRAH, 2005).

During this initial process control points positions that will be used το support the survey, are approximately selected. When TPS surveying is employed as the primary surveying methodology, main traverses should be designed in a way that makes them as few as possible, based on the premises that (a) control points need to enclose the features to be surveyed, and (b) be as close as possible to them. Additionally, angles between stations should not be too sharp so as to avoid errors.

During a reconnaissance survey, it is imperative to prepare a sketch illustration (croquet) of the area to be surveyed, so that the main landmarks in the area are described, such as existing buildings, streets, and fixed details. An indication of the direction of the north can be used in the sketch and the landmarks can carry additional information with some names noted etc. This sketch will be used as a basis for field notes recording.

It can also be useful in this regard to take some digital pictures for use in the planning of the survey and the preparation of the survey report for the project. Furthermore, these digital pictures can be used, when necessary, as evidence and proof of the existence or not of some facilities and other important structural elements (MoMRAH, 2005).

4.1.2. Existing Mapping Products in the KSA

Existing products can be provided by KSA authorities such as GASGI etc. The products include but are not limited to, Aerial and Satellite Imagery, Standard/Topographic and Cadastral mapping, Digital Elevation Models (DEMs), Hydrographic maps, etc. Their products can help in the process of planning field survey work and help in deciding the appropriate survey method for each work.

The designer/consultant appointed with the work of survey, when searching for available existing map data should not be limited to the following list, rather, he should conduct his own research for other available maps and surveying works in any relevant/concerned authority in the KSA.

4.1.2.1. Aerial photography

GASGI owns Orthophotomaps produced from Digital Aerial Photography procedures, covering several parts of the KSA.

- Orthophotomosaics in 1:50,000 scale with a resolution of 0.5 m. Each map covers an area of 15′ longitude×15′ latitude or the equivalent of 27 × 27 km.
- Orthophotomosaics in 1:25,000 scale with a resolution of 0.25 m. Each map covers an area of 7.5′ longitude × 7.5′ latitude or the equivalent of 13.5 × 13.5 km.

These Aerial products are using WGS 84 as the horizontal datum and the mean sea level is implemented using the old vertical Datum (Jeddah, 1969).

Products can be delivered in digital Geo Tiff format files on a digital production medium per user request or high-quality paper-printed products per user requirement.

4.1.2.2. Satellite imagery

Several satellite imageries are available throughout the Kingdom by all major imaging satellites, with images available at a cost through several providers, and at several sample dates. In Table 4-1 the GSD per satellite is summarized.

Table 4-1 Satellite Imagery and GSD

Imaging Satellite	GSD (m)
GeoEye	0.41
WorldView-2	0.46
WorldView-3	0.31
Pléiades Neo	0.30
QuickBird	0.60

4.1.2.3. Standard / topographic mapping

General and Standard / Topographic of the Kingdom of Saudi Arabia are provided to users through the CASGI’s site in digital form.

- Road map in scale 1:3,000,000 in both Arabic and English language. Road map contains urban and isolated populated centers, transportation networks, facilities, digital hill shading and sea depths, etc.
- Guidance maps for Cities and Urban, Cultural areas, including sacred sites, etc.
- General Map of the KSA in scale 1:1,000,000. Map covers the entire territory of the KSA and other neighboring countries in the Arabian Peninsula.
- International Land Borders of Saudi Arabia provided in shape file and GeoDB format.

- Topographic Maps of the KSA in scales 1:50,000 and 1:25,000 used for planning and engineering projects. A topographic map contains information on urban populations, isolated and rural buildings, transportation networks, utilities, road distances, etc.
- Digital Terrain Elevation Model of the KSA. Elevation values are placed in every 15' X 15' geographical cells with a consistent spacing every 30 m approximately..

4.2. Instrument Calibration

All equipment used in road project surveys shall be calibrated prior to the survey, at the earliest one year before the start of field survey work. Equipment should be also calibrated and tested from time to time and adjusted, if necessary, at least once a year. These tests may be carried out by repair and maintenance specialists or by the surveyor himself. In both cases, the test results should be identical to the maximum permissible errors indicated in the manuals of these devices, and the test documents should be attached to the survey work report. Standards applied for instrument calibration are ISO 12858, ISO 16331, and ISO 17123.

4.3. In Advance Preparation

Fieldwork should be in advance prepared by efficiently programming the next day's work and distributing responsibilities to the members of the survey team. Defining the appropriate procedures and methods depending on the project can assist to eliminate any unnecessary obstructions that may occur.

4.4. General Documentation Requirements

One of the advantages of current surveying instruments and technology is that it makes it possible for data to be automatically saved, organized, classified, and scheduled. Without this technology, choosing the conventional route is unavoidable. Either way, it is necessary to be aware of the following fundamental conditions for accurate recording and the proper arrangement of survey data and observations that raise their consistency.

4.4.1. Data Accuracy

If measurements, observations, subsequent computations, and drawings are not precise and accurate representations of the situation, they are useless. Therefore, it is imperative to take all reasonable steps to achieve the required accuracy without increasing or decreasing it. Decreasing accuracy may result in the failure and invalidity of the entire project while increasing the accuracy may result to increase costs and cause a delay to the completion of the entire project.

4.4.2. Data Integrity

It is necessary to verify the completeness of the collected information at the end of each fieldwork day before leaving the site. This supports the field work sequence between days without confusion, or waste of time, while at the same time, ensures avoiding unnecessary revisits due to incomplete measurements.

4.4.3. Legibility

To prevent any uncertainty or doubt regarding the accuracy of the written notes, numbers, or letters, it is essential that the words and figures in the field notes book be readable. This depends on the surveyor's expertise, experience, and precision as well as the quality of the paper, ink, and pens used.

4.4.4. Comments

Providing comments during a survey is essential for all subsequent processes of design and project implementation. Survey comments supplied can be consistent, necessary, and relevant.

Electronic data collection provides the ability to easily attach attributes to data, using the appropriate data collector software, which easily repeats the last recorded attributes to the next point. Moreover, data collecting software usually provides the ability to modify specific points or point properties in cases where such need arises, by allowing manual adding of specific values. Although this process is usually natural and easy to be achieved, often results to redundant data and high data collector memory utilization. This fact highlights the need for avoiding repetition of unnecessary comments attaching them to only the required observations. Moreover, irrelevant, or unused comment often create confusion to the final data users leading to errors.

4.4.5. Field Notes Recording

Field notes collected by crews using GNSS or Total Stations, robotic or otherwise, are often nowadays collected and kept in electronic format. Despite the use of electronic data collection built on surveying instruments, field crews should also record appropriate information in field books or other digital devices. This is not meant to duplicate the effort of the data collector, but to supplement with details, significant events, or corrections that need to be made to the electronic file and other such important aspects of a field survey.

As such, surveying field notes include a variety of data in digital and hard-copy formats of following data types:

- Handwritten or electronically recorded notes.
- Electronic records from measurement instruments. These may be in their original format, or an equivalent interpretable format.
- Records that are corrected, adjusted or rectified by good practice survey methods to allow survey calculations to be undertaken.
- Documentation of the correction, adjustment or rectification methods and results.
- Imagery including but not limited to, photography and video.

Under this perspective, the end product of a field survey is a set of field notes.

Field notes exist to document the survey work performed. Their existence is of great importance; they even, in some cases, obtain formal trait and become legal documents (for example, in boundary and construction surveys). As such, removing notes or erasing them, is not allowed in field books. If alterations/corrections should be made to the notes, a line shall be drawn through, indicating the error (without obscuring it) and the correct value or information should be written adjacent to it (Illinois DOT, 2015).

The following guidelines should be considered when using field notes (Illinois DOT, 2015); (MoMRAH, 2005); (NSW BOSSI, 2018) :

- Field notes should be prepared in a neat, precise, complete, and readily intelligible manner in accordance with the usage of surveyors.
- All facts, readings and observations shall be recorded immediately after they are ascertained.
- The full names of all crew members, who participated in the survey, shall be recorded on the used means.
- The date that the work recorded in the field notes was performed shall appear on the field notes. Survey field notes should also include the project number and title, the project identification, and the project location.
- Data entered in the field notes should not be crowded. Using data collector specific additional notes or use of additional pages is preferred.
- Sketches should be kept plain and uncluttered.
- Numerical values should be recorded in such a way that they specify the degree of precision to which a measurement is taken. For example, rod readings taken to the nearest 0.01 m should be recorded as 2.50 m, not as 2.5 m.
- Sketches are suggested, especially when it is impossible to arrange the information effectively. It is important to note that when drawing sketches, scale is not necessary because it is possible to exaggerate some elements while also reducing others. In order to grasp and transform the data from the field book into precise and understandable charts and maps, it is important to show the subtleties and distinct points clearly.
- Use of photography with electronic means of data collection for sketching is encouraged.
- Sketches drawn by electronic means may be drawn in scale, using available satellite or aerial imagery background, when recorded data density is rather uniform.
- Explanatory notes should be used to supplement numerical data and sketches.
- Field notes shall include the nature and position of any survey mark or monument found by the surveyor and the nature of any survey mark placed by the surveyor.
- The datum line of the survey and the origin of the orientation adopted should be clearly indicated in their field notes.
- Names of estates, houses, roads, rivers, creeks, lakes, and house numbers shall be recorded by the surveyor in their field notes.
- When using handwritten field notes it is advisable to use a sharp, fairly hard pencil (3H or 4H). In this case, a field notebook shall be used. It is also advised to use idiomatic symbols (when conditions suggest so) to reduce writing, provided that there is an accurate table of these symbols along their meanings.
- Before leaving the site, it is advisable to conduct a short and thorough check, review the recorded information, go around the area that was surveyed, and ensure that everything has been checked.
- Surveyor shall personally sign and date each page or sheet of the field notes, for all surveys that have been performed by the surveyor personally or under its supervision.

In the case of electronic means, a digital signature shall be used to sign the respective files, by any available electronic method, e.g., printing data to pdf and sign the resulted pdf file, or sign a compressed zip file including the respective field note and data collector data. Before signing field notes and electronic data, surveyor shall be ensured of being satisfied that all relevant information is accurate.

4.4.6. Field Notes Management and Submission

Survey field notes general lifecycle from capture, to use, and archiving is presented in Figure 4-1. Given the importance of field notes, consideration needs to be made regarding their recovery at an unknown point in time after archiving. The field notes should be recoverable, to the extent that they can be used to verify and authenticate any results reliant on the captured data. Digital data should be archived in both the raw capture format, and a readable format containing all relevant field data.

Field Notes should be kept indefinitely as surveyor may need to refer back to them when answering requisitions, investigations undertaken by the concerned authority or a legal case. The following remarks about filed notes management should be considered:

- Field notes, should be kept in archives with indices and cross-references, and all other information and documentation relevant to those field notes
- An electronic copy, additional to the reduced data, of all raw measurement data shall be retained, including angular and distance observations regarding TPS and as well as raw GNSS observation data regarding GNSS methods.
- Scanned copies of hard copy field notes are an acceptable format to store field notes in the long term. Scanned copies should be kept in a format that is universally recognized (such as PDF, JPEG, TIFF). Field notes should be scanned at a resolution that ensures the integrity of the original field notes is not lost.
- Archived electronic field notes should be ensured that they can still be opened with the latest hardware and software when transferring data. This may involve converting old data into new formats which can still be opened and read at a later date.

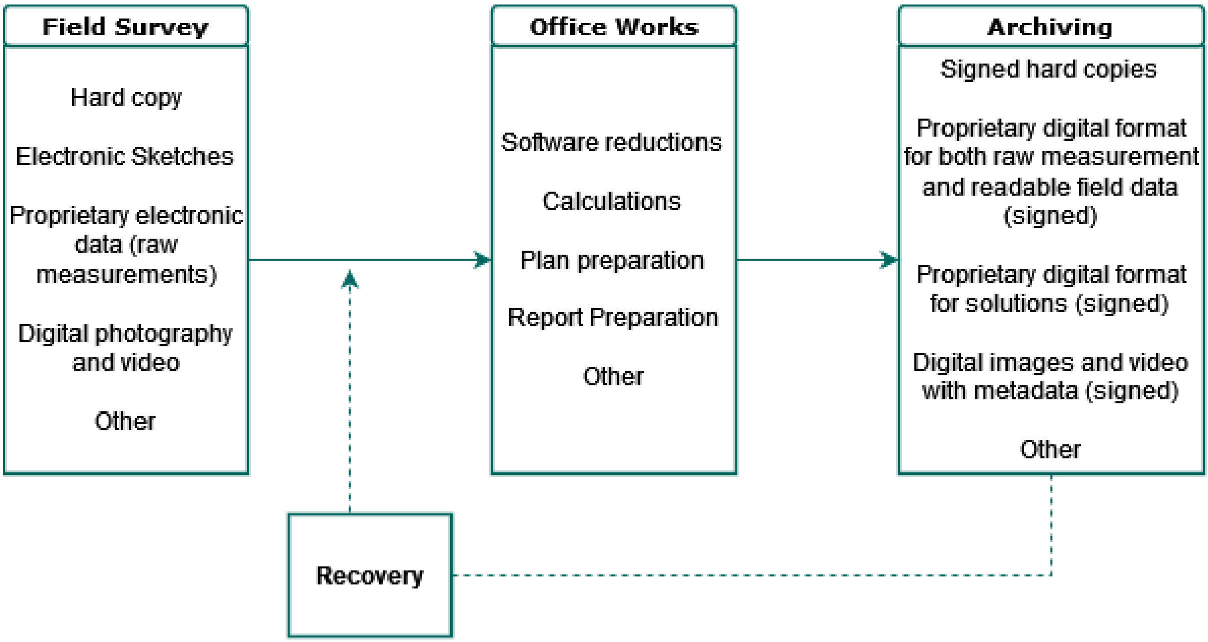


Figure 4-1 Field Notes Lifecycle

4.4.6.1. Hand-written notes

Among the various types of field books available when using hand-written notes, the most commonly used is the 114 x 184 mm size. Each field book prepared by or for the client should be compiled into field books of this size with canvas or leather imitation so that there is a uniform result.

Sometimes, it is necessary to use duplicated field books. The duplicated books are required mainly when it is imperative to send a copy to another agency, authority, or the client. Under such circumstances this type of books can be used to retain a record of the notes after the penciled field books have been turned over to the other party.

Each field book should have a notation on the first leaf requesting that if it is found, it should be returned to the concerned Client Engineer whose name and contact details are given inside the field book. A few guidelines regarding field note indexing are presented below:

- A legend map that demonstrates the survey site could be pasted on the inside of the front cover to assist the user of the survey book. On the same page, the date of survey start should be written.
- The second sheet of the field book should bear the names and positions of the survey crew.
- Pages should be numbered in the upper right-hand corner throughout the book
- Additional data that may be added later could be fit on blank pages that have to be intentionally left blank in the front and at the back of the book for.
- The date and weather conditions should be provided on the page that begins each day's work.
- When the notes in the field book are completed, an index should be placed on the first of the blank sheets that were left in the front of the book.
- If the book contains level notes, they should be checked in the field and a note placed on the very last page indicating the name of the checker and the date.

- On topography notes, the curve data should be checked, and a similar note made.

Information on how the survey began should be placed on one of the early pages. The use of a paragraph and explanation sketches is encouraged there. Furthermore, information about the datum used and whether the stationing is new, a continuation of a previous or arbitrary one, should be included.

4.4.6.2. Submitting

The consultant should submit if required, hard and soft copies to the Client during or at the completion of the field surveying work. The paper document should be bounded on a volume that contains all collected data on A4-size paper. The name of the consultant should appear on all paper documents submitted to the Client. Electronically collected survey data will include both raw and processed survey data.

4.5. Field Communications

It is important that the survey team(s) communicate adequately and have good relationships with every person involved in properties being close or likely to be affected by the project. Towards this goal, it is significant that communications of the survey staff are performed formally; hence the importance of a property owner contact form and the notion of right of entry arises.

4.5.1. Property Owner Contact

At least one week before starting any field survey activity affecting private property right of entry is needed, letters from the Responsible Agency or any other Client shall be mailed to affected property owners. Before entering the property, the survey crew should focus on establishing a good relationship with property owners and obtaining their approval. It is desirable contacting in person so as to clarify to property owners the scope of the survey, the reason why entry is required, and all other activities involved. The Property Owner Contact Form (Figure 4-2) may be used to document conversations with property owners (DMAT, 2016).

4.5.2. Entering Private Properties

Each private property owner should provide the right of entry prior to executing any type of surveying on his/her property. In order to document the right of entry, Client will provide relevant forms, such as shown in Figure 4-2, that has to be signed by each landowner before attempting entry to his/her property. In cases where signed forms cannot be acquired timely, a verbal right of entry may be obtained; nevertheless, every attempt to secure a written right of entry should be made before the survey is completed. Details about the duration that the control target will be on the property's ground and the responsible personnel can be included in the form. Agreement may also include additional details such as specific times for access to the property or conditions on notification before entering the property.

Surveying staff should demonstrate courtesy when entering private property, even in the case where Client has the legal right to enter private property. The surveying crew should properly notify property owners in advance, especially when their land is being occupied for an extended length of time or when a large portion if their property is involved in the survey. Survey personnel entering private property should use discretion and respect for the owner's property. Specifically, personnel should:

- Avoid littering: Obtain permission before setting any survey points, aerial targets, etc. Always clean up the site after surveying work on the site is finished.
- Seek permission for vehicular use and acceptable routes of travel when entering planted fields. Minimize crop damage and try to use the same tire tracks when leaving.
- When passing through a gate, if it is shut, always make sure it is shut again after passing through it.

If a property owner refuses to provide the right of entry, or even retreats from his provided permission, surveying staff should comply and report the event to the client/concerned survey authority. The surveyor in-charge should make negotiations with the property owner to obtain right for entry/re-entry. In the event that no permission is granted, legal action may be taken.

Property Details:					
Name(s):					
Address:	*P.O. Box:*				
City:					
Map No.:	*Parcel:*				
Deed Book:	*Page:*				
Remarks:					
Project No..:					
Description:					
Person Contacted:	*Owner*			*Renter*	
The following exists on the property:					
Wells	*UG Utilities*				
Septic Tanks	*Graves*				
UG Fuel Tanks	*Property Corners*				
Water Tanks	*Palm Trees*				
High Water Marks	*Building Types*				
Irrigation Lines	*Fences*				
Well Dug By:	*For:*			*Date:*	
Comments:					
Sketch:					

Figure 4-2 Property Owner Contact Form (DMAT, 2016)

4.6. Safety During Survey

Surveying projects can take place in various locations with different environments, characteristics and conditions (inside buildings, roadways, highways etc.) No matter what conditions apply it is important to create a safe environment for the employees, pedestrians, and drivers for the abrupt and swift completion of the project at hand. Education and training in the same manner, that road safety engineering is the most efficient means of preventing accident causes (dangerous conditions) from the surrounding environment, as road safety education is the most efficient means of preventing unsafe activities. Employees should build safe attitudes and relevant knowledge through sufficient training. Safety consciousness established through education will be strengthened by supplementary teaching in safe working habits, practices and abilities. Accident prevention can be minimized by educating and creating a safe work environment. Below are presented Road and Surveying Crew safety measures, in order to communicate, educate and inform the concerned authorities and personnel about safety measures that need to be followed while carrying out projects.

4.6.1. Road Safety Measures

Road safety is of high importance in land surveying operations and needs to be dealt accordingly (SHC 305 Highway Facilities and Utilities Design – Work Zone Design). Both survey personnel and road users/persons being close to survey works should receive the maximum possible level of safety; as such, survey parties should carefully utilize their equipment, and thoroughly follow all procedures available. The major factors impacting the protection to be used in each situation include (FDOT, 1999) and (SHC 305 Highway Facilities and Utilities Design – Work Zone Design):

- Speed.
- Volume of traffic.
- Highway geometry.
- Duration of operation.
- Exposure to hazards.
- Weather (or road conditions).

When working in the right-of-way, whenever they are exposed to traffic, such as when in driveways, parking lots, construction sites, etc., or whenever the on-site supervisor deems it essential, employees shall wear high-visibility safety vests or yellow and orange raincoats. Additionally, the safety vests need to be reflectorized for nighttime operations. The reflective vest shall be worn as the outer layer if raingear or another outer garment is worn (Florida DOT, 1999).

Personnel shall be instructed concerning the importance of staying alert and being courteous to the road users. The person in charge of the survey party has the responsibility to inspect that the necessary safety precautions are properly taken. Surveyors should exercise extra caution while around large and swiftly moving machinery, particularly when on haul roads and when around machinery with poor driver visibility.

When working on or close to the road, appropriate traffic signing shall be applied as the lowest measure of road safety. There are two categories regarding road signing for surveying works: warning signs and channelizing devices (SHC 305 Highway Facilities and Utilities Design –

Work Zone Design). Before beginning any survey work on pavement or within 5 m of the edge of the travelled path, warning signs shall be put in place. They should be utilized every time survey staff work in a busy area. The importance of changing warning signs as the work advances is underscored by the fact that surveyors are continually moving along the route. These devices shall be switched, removed, or covered whenever work stops, even at the end of the day while conducting surveys. This straightforward process will shield motorists and surveyors from a wide range of potential issues. When the nature of the activity changes to the point where a certain sign or other warning device is no longer necessary, the sign or device shall be turned, removed, or covered, and if necessary, replaced with the proper device. The most common signs are "Road Work TW 50-1," "Work Zone Ahead TI 1-1," "FLAGGER AHEAD TW 50-3," (SHC 305 Highway Facilities and Utilities Design – Work Zone Design). Amber lights can be used to warn oncoming vehicles of workers on foot or nearby operations. When there is no risk to the workers or bystanders, avoid using them. Warning lights lose a lot of their effectiveness when they are misused or used excessively as it could unnecessarily blind or divert traffic (FDOT, 1999).

At locations where surveyors may be present for an extended amount of time, such as instrument setups, protective vehicles can be particularly crucial. Between the moving work site and the approaching traffic, protective vehicles might be parked. A shadow vehicle, which employs an attenuator and accompanies a survey operation moving in the direction of traffic, is an additional option. The positioning of the protected vehicles should make them effective traffic barriers. Keeping a safety vehicle close enough (but not too close) to provide true physical protection for workers while avoiding putting them in danger.

Regarding channelization devices, these are one of the following types: cones, vertical panels, plastic drums, and barricades. Vertical panels and cones are to be used only when a portable device is needed for lane closure operation that will last only one day or when the more durable barricades are not available. Barricades should be used whenever appropriate. They are, however, difficult to carry and stack. Cones, being orange in color, are used to channel traffic through and around a work area (SHC 305 Highway Facilities and Utilities Design – Work Zone Design).

The surveyor may occasionally need to split or close off traffic. Cones are employed to do this. Plastic drums should only be used if better options are unavailable. Due to their size, drums are not used on a daily basis.

Where signs and channelization devices are used, they should be placed prior to beginning work and at locations having adequate approach visibility. Field conditions will also control the actual placement of signs and channelizers. Personnel using stop-slow paddles are positioned as needed to provide the best direction of traffic and safety for working personnel.

Trucks with top-mounted flashers may be used to provide additional protection for personnel by blocking the lane or parking on the shoulder. For safety considerations, the involvement of the police may be required. This is particularly important before start of the surveying work while implementing safety procedures including the installation of temporary warning signs (Road Work TW 50-1), safety cones, etc., and after completion of the surveying activities during dismantling of safety equipment (SHC 305 Highway Facilities and Utilities Design – Work Zone Design).

Before the start of any survey work, prior approval of all safety procedures has to be obtained from the concerned authority.

4.6.2. Surveying Crew Safety Measures

Besides the Road Safety measures is important to create and offer a safety environment and conditions for the crews and personnel working on site. Employee safety should be taken of utmost importance because a safe working environment offers the ability to complete the project as fast as possible and with minimum complications.

Safety meetings can occur prior each project to ensure the adequate protection and education of the employees. Use of clothing that provides adequate protection is advised. To begin with, during any operations where there is exposure to hazardous situations, all staff shall wear the proper personal protective equipment. Therefore, high-top leather work boots that offer the most protection shall be worn by survey employees (ODOT, 2013).

All employees shall wear approved protective helmets while working or visiting areas designated by the resident engineer as "Hard Hat Areas." These helmets will undergo regular inspections and shall be replaced right away if found to be defective. Helmets should be worn whenever it is judged required, including when working below overpasses. Protective goggles/glasses, safety vests, dust masks, hearing protection, gloves and other auxiliary equipment can be used if necessary. Crews should be fully prepared for the local weather and climate. In conditions of heat stress and sun exposure employees should follow preventive measures such us hearing lightweight clothing, head coverings, frequent rest breaks etc. When conditions such as extreme temperatures don't ensure the safety of the employees work suspension should be applied until safety conditions are restored (ODOT, 2013).

Surveying crews when on foot and near or on a driveway, should work facing oncoming traffic (when conditions permit). If its unable to work facing traffic, a co-worker can assist as a "spotter". It is advisable to face traffic from any work site within the right-of-way. When crew members working in areas with traffic such us highways, driveways etc. they should not make sudden moves that could confuse drivers and cause harm or injuries to workers, passerby, drivers etc. Surveyors, when working on or close to roads should avoid interrupting traffic as much as possible. One way to successfully not interrupt the traffic is to apply offset lines. Especially, on high traffic and high speed highways is preferred to avoid , or minimize the need of road crossings. If deemed necessary to cross, crew members should at least wait for a natural break of traffic. Furthermore, barriers, caution tapes, truck mounted attenuators etc. can be used to protect surveying crews during measurements (SHC 305 Highway Facilities and Utilities Design – Work Zone Design).

All workers should be aware of the safety innovations, including tools and methods, in order to conduct surveys safely. Continuous safety monitoring is necessary. All workplace safety violations should be fixed right away. All adequate safety equipment should be in stock.

Animal hazards can be avoided rather efficiently if crews assume that all animals in the area of work are considered hostile and potentially dangerous. Members of crews should not approach any animals domesticated or wild to pet them. Always secure that owners have contained and secure possible hostile acting domesticated animals prior entering a work area or an enclosed area of importance to the completion of the measurements (FDOT, 1999).

Before the start of any survey work, prior approval of all safety procedures has to be obtained from the concerned authority (SHC 305 Highway Facilities and Utilities Design – Work Zone Design).

5. Preparing Maps

Survey work, regardless of the instruments and methodologies employed, ends with identifying the features that were observed in nature on maps with certain drawing scales according to the nature of the project under consideration, and its lifecycle's phase. As such procedures such as feasibility study, require the production of small-scale maps, while preliminary, and final/detailed design phase require larger scales, according to the details presented in Section 3.2. Regardless of the survey method and road project's lifecycle phase under consideration, final maps produced by survey works, shall follow the following specifications regarding feature representation, horizontal and vertical details, map outline on which survey data are displayed and quality control procedures. Features displayed on maps shall follow the accuracies posed in Section 2.7.

5.1. Feature Representation

Features on the map, are represented using three basic symbols: point, line, and polygon.

- Point: It is the symbol used to denote the features of small size or the features that the scale of the drawing does not allow to show them in their form, such as: control points, elevation points, wells, electricity poles, lightning pillars, traffic signs, trees etc. Point features are usually symbolized with markers as circles, rectangles, and other icons of fixed dimensions.
- Line: Line strings / Line features represent the shape and location of geographic objects such as centerlines, roads, railways, streams, rivers, public utility lines etc. Basically, line features are connected point features. Lines have length but no width. Line features are symbolized with continuous, dashed, dotted lines, of appropriate width, color etc.
- Polygon: Polygonal features represent the shape and location of homogeneous feature types, such as countries, soil types, properties, lakes, vegetation boundaries, plots of land, farms, green areas, sandy areas, quarries, building footprints. etc. They consist of one external and several internal rings (boundaries) which are closed line-strings. Symbolization in polygonal features deals with the symbolization of their boundaries, being closed line features, and their interior. These symbols appear in different positions and designs such as: size - shape - pattern - direction - position - texture - color.

Feature visibility and representation according to the previous feature types is also dependent on the respective map's scale; features displayed as polygon types in large scales could be represented by line or point features in smaller scales.

Features shall be categorized in layers, displaying various types of buildings, roads, and other features. Table 5-1 displays the number of layers used by feature type regarding main features. Appendix A contains the details regarding 310 layers included in the Feature Codes of Topographic Maps (MoMRAH, 2005). The SHC urge the user of this code to follow any updates MoMRAH in these layers and be aware by any new products introduced by the GASGI.

Table 5-1 Main Features and Number of Layers (MoMRAH, 2005)

Section	Feature Type	Number of Layers
A	Distinctive Areas	31
B	Buildings	47
C	Constructions	23
D	Transport	92
E	Facilities	35
F	Determinants	18
G	Marine Features	46
H	Elevations	18
I	Plants	36
J	Margins	19
K	Cartography	8

5.2. Horizontal Details

Final maps produced by survey works shall show, as a minimum, the following horizontal details (MoMRAH, 2005):

1. Buildings and Establishments: Permanent buildings with an area greater than 6 mm^2 on the scale of the map should appear with their outer limits. Smaller buildings can either be generalized to point features or removed as appropriate. Ruins, partially demolished buildings, buildings under construction, and other buildings should be shown in dashed lines, ignoring small shape turnings that cannot be represented within 0.75 mm on the scale of the map.
2. Boundaries: Walls, fences, and similar field boundaries must be indicated on the map with single lines representing the natural axis (center) of the boundary. In the case of fixed property boundaries, the line shall represent the outer edge of the boundary. If the walls are thicker than 0.50 mm on the map scale, should be indicated by double lines in their true locations.
3. Roads, Railways: Road edges, side lanes, or boundaries of sidewalks, bridges, tunnels, and retaining walls shall be shown on the map. Dirt and pedestrian roads shall be indicated either by continuous dashed lines, by thinner lines, or by sweeping edges, whenever they are clearly defined and of sufficient width to be signed on the scale of the drawing of the map. Railroads are indicated either by line width or by special symbols.

4. Utilities: Towers, electricity poles and visible utility elements shall be shown when appropriate to the scale of the map.
5. Waterways, Drainage Channels and Coastal Features: Wells, Wadis, and canals shall be shown with double or single lines, proportionally with their width and scale; valley bottoms are indicated by dashed lines. Lakes, ponds and seacoast line should also be shown.
6. Topography, Vegetation, and Land Use Classification: Terrain features such as rocky outcrops, tall boulders and sand dunes shall be included in the map. Vegetation cover and land use features should be specific only to the basic categories of palm gardens, forests, crops, and farms. Individual large trees shall be displayed on their own. Other nonstandard features, such as cemeteries, quarries, sports fields, parking lots and recreational places shall be also displayed.
7. Names and Comments: The names of places and regions, cities, villages, natural features, streets, large buildings, road divisions, and visible surface facilities shall be indicated as taken from existing maps or according to what was collected during fieldwork.

Maps on a Scale of 1:1,000 and Larger shall further, as a minimum, contain:

1. Utilities including manholes along with their type, inlets types, handholes descriptions, types of different chambers, lights, traffic lights and poles of any type, irrigation devices, gas accessories, etc.
2. Water bodies and streams elements should be further shown in the map and displayed in a way that help determining breaklines used for correct DTM construction. Such elements include toe and top of slopes, ditch bottoms, curb gutter lines, etc. Moreover, maps of this scale should contain any linear strings compiled along the breaks or changes of grade in the terrain, identifying terrain discontinuities such as ridges, toe of slopes, or any abrupt change in the existing surface. Section 17.1.3 and Figure 17-3 provide further details.
3. Road equipment and highway elements, such as bike lines, landscape features, vertical and horizontal marking and Vehicle Restrain Systems with the definition of their material (i.e. steel, concrete). Different railway components should be also included, such as top of rail, edge of ballast, toe of slope of ballast, edge of sub-ballast, etc.
4. Additional information including mailboxes, telephone booths, benches, fountains, fire extinguishers, light poles, large arched entrances to buildings that are more than two meters wide and visible from public areas, and gates that are more than 2 m wide.

5.3. Vertical Details

Vertical details are represented in maps by spot elevation points and contour lines. Typical contour intervals by scale are given in Section 2.7. Contour lines shall be derived using the respective DTM created as described in Chapter 17 of this Volume. Contour lines shall be categorized into major and minor. Major contours should be defined every five contours and represented by thicker lines than the minor contour lines. Minor contours can be omitted on steep slopes if their convergence is less than 1 mm on the map scale.

Spot elevations should be at the following places (MoMRAH, 2005):

- Between normal contour intervals at 50 - 100 mm from each contour on the scale of the map, provided that such distances exist.
- At sites of land protrusions such as the tops of hills.
- At flat sites.
- At depressions.
- Along road axes and at intersections.
- Along the fundamental changes in the gradients in the rise or fall.

5.4. Map Outline

Map outline shall meet the following requirements (MoMRAH, 2005):

1. Final map shall include coordinate grid, control points, horizontal features, contour lines, elevation points, labels, etc.
2. Map must include sheet number, map scale, North sign, coordinate system used, project name, map index containing adjacent sheets, date of survey, contour intervals and heights. Comments or footnotes must be written clearly. The grid lines are displayed with continuous lines every 100 mm at the map scale.
3. Final map sheets shall be plotted directly from digital data on dimensionally stable polyester transparencies with a thickness of at least 0.15 mm.
4. Map shall include a legend, with the point symbols, line types, line colors and widths. The sizes and shapes of the print must be identical to what is displayed in the relative map sheet.
5. Map index, numbering, and plate size must conform to the map division system so that the survey area is covered with a sufficient number of economically appropriate sheets, which in their total size shall not exceed the size of A0, i.e., 841 x 1,189 mm, and without repeated copying of the details in the overlaps. The actual drawing area must be placed within the limits of the standard formats for the plates provided by the respective project's owner. The actual area for drawing on a map on a scale greater or equal to 1:2,000 should be 1,000 x 750 mm; the same area for scales less or equal to 1:2,500 shall be 800 x 600 mm. Table 5-2 summarizes this information.

Table 5-2 Dimensions and Map Areas According to the Scale (MoMRAH, 2005)

Map Scale	Dimensions of Actual drawing (mm)	Map area (km^2)	Dimensions (km)
1:500	1,000 x 750	0.1875	0.5 x 0.375
1:1,000	1,000 x 750	0.75	1 x 0.75
1:2,000	1,000 x 750	3.0	2 x 1.5
1:2,500	800 x 600	3.0	2 x 1.5
1:5,000	800 x 600	12	4 x 3
1:10,000	800 x 600	48	8 x 6

Map Scale	Dimensions of Actual drawing (mm)	Map area (km^2)	Dimensions (km)
1:20,000	800 x 600	192	16 x 12
1:25,000	800 x 600	300	20 x 15

6. The following data shall be included in map margins: Symbol legend, title field, project name, project location, contracting authority, project number, sheet name, map scale, map type, key displaying neighboring sheets, preparation date, geodetic reference and signatures required.
7. Names displayed on the map should regard important features. Type and size of names included in the map should be selected according to the respective feature categories. Examples of the named features categories in urban areas are region, city, and administrative boundaries; mosques, schools, and shopping malls; parking lots, public squares, distinguished places, and cemeteries; longitudinal and water features; roads, main and secondary streets, and railways.

5.5. Quality Control Procedures

Map compilation requires certain control procedures that guarantee its quality. Survey works, being linked to the rights of many citizens, and serving as the basis for the implementation of development projects, are one of the most important works on which strict quality control procedures have to be applied. In all cases, the following steps shall be taken to control the quality of produced maps:

1. The procedures for controlling the quality of surveying work shall be undertaken by a surveying engineer, head of surveyors, or the respective specialized unit according to the organizational structure of the authority in which the work is carried out.
2. Ensuring that all the methodological steps described in this Volume are implemented, and not relying on the final accuracy that has been reached, as this may be a pure coincidence.
3. Examining the accuracy of the observations and making sure that the average squared error is less than the permissible.
4. Examining the documents, cadastral records, and survey reports, and making sure that they were prepared correctly and systematically.
5. Examining the map produced in the survey work, ensure its quality and absence of usual drawing errors, including the presence of unclosed polygons, overruns, and shortcomings in the intersection of lines, the presence of protrusions in the map, incompletely defined or unconnected features, or being outside their groups and this is confirmed if the map is produced in digital format.
6. Marking the survey work indicating that it has passed the quality control procedures.

An independent quality control unit should be designated with the task of quality control reviews, following the workflow presented in Figure 5-1 (MoMRAH, 2005).

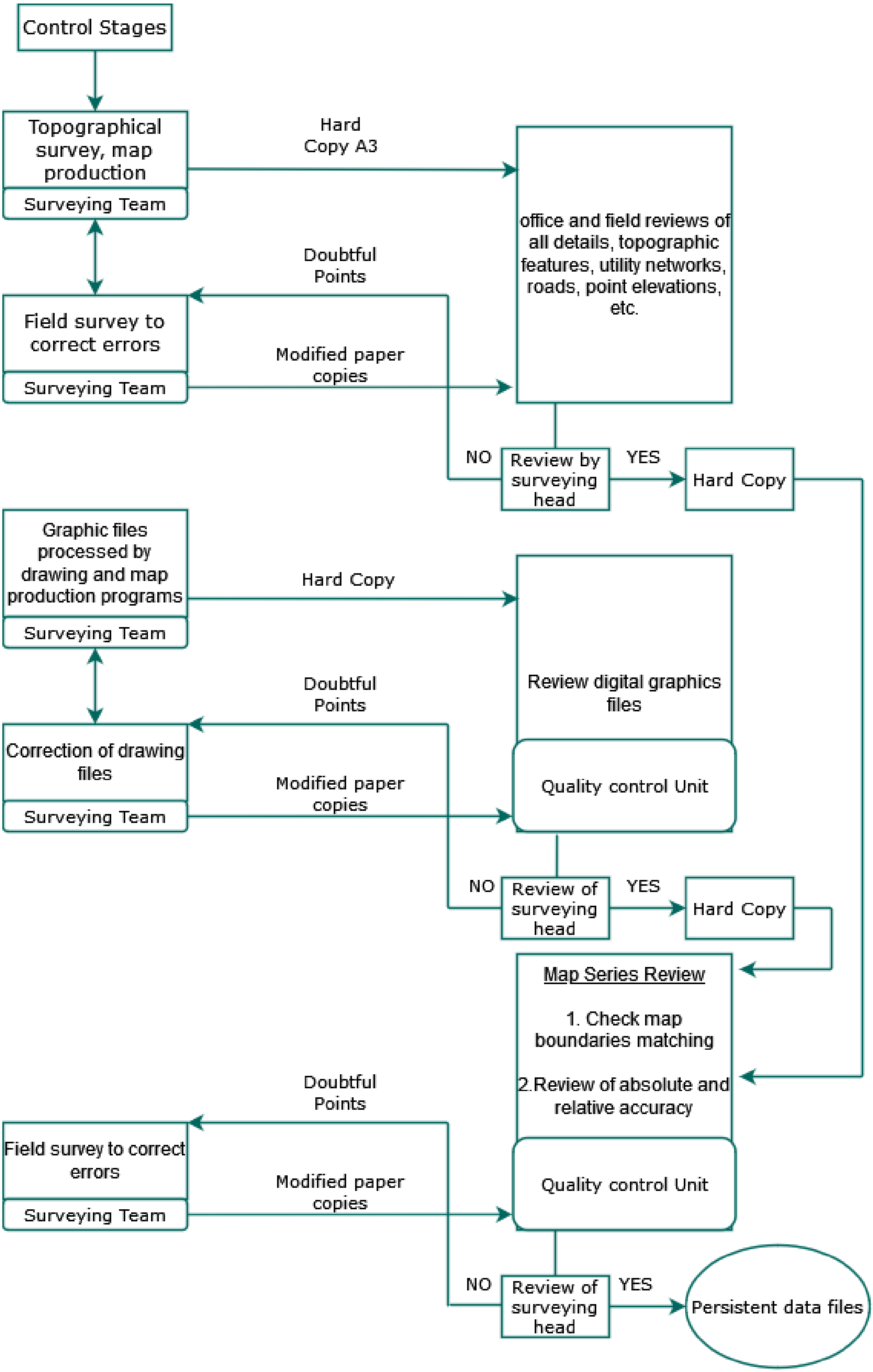


Figure 5-1 Flowchart for Quality Control of Topographic Maps (MoMRAH, 2005)

6. Data Delivery and Formats

Data delivery formats refer to procedures of data transferring from consultant to client regarding the deliverables of each survey work stage.

6.1. Survey Data and GIS Data

All data collected during surveys describe locations and elevations of natural and man-made features displayed on a map. Nowadays, mapping is supported by GIS and design software that allows displaying, symbolizing, and rendering data on maps. Therefore, data collected are plotted in common GIS file formats to develop topographic maps and three-dimensional (3D) terrain models. Survey and mapping data are categorized into the following:

- Raster: In its simplest form, a raster consists of a matrix of cells (or pixels) organized into rows and columns (or a grid) where each cell contains a value representing information, such as temperature. Raster representation is used to map continuous changes in natural phenomena. Common raster files include digital aerial photographs, imagery from satellites, digital pictures, scanned maps, DSM, and DEM files. Popular raster formats include JPG, TIFF, BSQ, IMG etc.
- Computer Aided Design and Drafting (CADD): CADD in the context of mapping, is the use of computers to aid in the creation, modification, analysis, or optimization of a drawing containing a map. CADD files are used in the production of maps, transforming raw survey data, i.e., field notes, into actual maps. CADD files contain raw survey data and surveyed features, along with their stylization and all other outline and descriptions, e.g., sheet number, map scale, project name, contained in the final map. CADD files classify features into layers, support the notion of Model (Design) vs Sheets, and may include (reference) raster files in them being used as base maps. Popular CADD formats include AutoCAD drawing file (DWG), drawing exchange format (DXF) and MicroStation design file (DGN).
- Vector GIS: Vector GIS formats include spatial features along with attribute data that can be used to be displayed on a map. Vector GIS formats may contain several layers (feature classes), with each one having several attribute data (data columns) attached to it. Under this perspective, vector GIS formats may also include database systems with spatial data support. Vector GIS formats do not contain any information about the stylization used when displayed on the actual map; therefore, vector GIS data serve only as a basis for producing maps. Popular vector GIS formats include SHP, personal and file geodatabases and geopackage.
- Map GIS: Map GIS formats include all necessary information about the stylization used to display vector GIS and raster formats on the actual map, along with other map sheet information displayed outside the actual map area. Given that map data formats only contain reference to the actual data, are often provided by the project’s owner, or can be developed independently and prior of completing survey works. Map GIS formats are specific on the software package used. Popular formats include MXD, QGS, GWS etc.

- Other: Other formats including raw and processed survey data files (field note files), pointcloud files, DTM-specific files, as well as final map files and report files, presented in Portable Document Format (PDF).

In the following paragraphs, delivery specifications are provided per file format category.

6.2. Raster Formats

Raster formats are used to deliver orthophotomosaics, orthorectified satellite imagery, DSM, and DEM data. The following points should be considered regarding raster data delivery:

- Imagery raster data shall be delivered in GeoTIFF data format, using 38-bit bands (true color) and JPG Compression at 90 %. Georeference shall be included in the actual GeoTIFF file.
- DSM and DEM raster data shall be delivered in 32-bit floating point GeoTIFF format. Georeference shall be included in the actual GeoTIFF file
- The pixel size regarding imagery data (orthophotomosaics and orthorectified satellite imagery) should equal the respective map horizontal RMSE. For example, according to Table 2-1 for a map of 1:1,000 scale, the respective orthophotomosaic file should have pixel size of 0.25 m.
- The pixel size regarding DSM and DEM files should equal 4 times the respective map horizontal RMSE, e.g., according to Table 2-1 for a map of 1:1,000 scale, the respective DSM file should have pixel size of 1.00 m.

Table 6-1 Raster Data Tile Dimensions According to the Scale

Map Scale	Imagery Size (pixels)	DSM/DEM Size (pixels)	Tile Dimensions (km)
1:500	8,000 x 6,000	2,000 x 1,500	1 x 0.75
1:1,000			2 x 1.5
1:2,000			4 x 3
1:2,500			5 x 3.75
1:5,000			10 x 7.5
1:10,000			20 x 15
1:20,000			40 x 30
1:25,000			50 x 37.5

- Raster data should be delivered splitted into tiles of dimensions provided by the above Table 6-1, at a grid of these dimensions starting at coordinate's system origin (0,0). Tiles should be named after X and Y coordinate of their lower left point, using the five first digits of X and five first digits of Y formatted with leading zeros to have a length of 7 digits. As such tile names should have a length of 10 digits in the form of [XXXXX][YYYYY]. For example, for a tile with its lower left coordinate located at 500,000, 4,000,000, the X part is “05000” and the Y part is “40000” and the total name is “0500040000”.

6.3. CADD Formats

CADD files are the most common deliverables of surveying and mapping works for road projects, especially in the phases of preliminary and final design. The following points should be considered regarding CADD file delivery:

- CADD files shall contain all map features surveyed classified according to the feature layer specification of Appendix A for scales of 1:1,000, or following MoMRAH’s feature layer specification for different scales.
- Each feature layer shall contain only data of the specific feature type (point, linestring, polygon). For example, spot symbol block references shall not be contained at any linestring feature layer.
- CADD files should, in addition to the surveyed map features, contain raw survey data, even though these data are not displayed on the final map.
- Map features shall be displayed in the CADD file model, while sheet outline and descriptions, such as, titles, scale, etc., shall be displayed in the drawing’s sheet (layout)
- Symbolization provided for feature layers in Appendix A or similar shall be used when no basemaps are employed. When imagery is used as a basemap, feature symbolization shall be adjusted according to the basemap’s color scheme after consultation with the client.
- All external files used in the main CADD file for map compilation shall be delivered additionally to the main file. These files include imagery, fonts, line types, plot styles, external references, etc. Most CADD software provides tools for collecting such external files used (e.g., e-transmit in AutoCAD).
- CADD files shall be delivered in DWG, DWF, DGN or DXF file formats. Acceptable file formats for survey data are controlled by the software packages used by the client, therefore, specific file formats may also be identified by the client.

Table 6-2 Topological Rules Regarding Single-Layer Validation

Feature Type	Applied Topological Rules
Point Geometry	must not have duplicates. must not have invalid geometries. must not have multi-part geometries. must not have empty or null geometries.

Feature Type	Applied Topological Rules
Line Geometry	must not have dangles. must not intersect. must not have duplicates. must not have invalid geometries. must not have multi-part geometries. must not have pseudo-nodes. must not have collinear geometries. must not have empty or null geometries.
Polygon Geometry	must not have duplicates. must not have gaps. must not overlap. must not have invalid geometries. must not have multi-part geometries. must not have pseudo-nodes. must not have empty or null geometries.

6.4. Vector GIS Formats

Vector GIS formats include spatial features along with attribute data that can be used to be displayed on a map. Vector GIS data formats are not frequently employed in surveying and mapping for road projects. Nevertheless, they are suitable for several deliverables, especially in cadastral surveys for road projects (Chapter 18). Contour data also produced automatically from DSM/DEM raster formats are provided into vector GIS formats. The following points should be considered for vector GIS file delivery:

- Vector GIS data should be delivered in Geopackage file format. Shapefile, personal, and file geodatabase can be also accepted.
- Several special topological rules should be applied and validate the quality of the features included in the given set. Such rules applied on a sole layer are described in Table 6-2, while Table 6-3 summarizes such rules applied over combined layers. For example, a layer containing point geometries could be checked if it is covered by polygons in a polygon geometry layer. The topological rules of Table 6-2 and Table 6-3 should be checked prior to the file delivery, given the context of the specific layer. These rules shall be checked using GIS Software.

Table 6-3 Topological Rules Regarding Combined Layer Validation

	Point Geometry	Line Geometry	Polygon Geometry
Point Geometry		must be covered by must be covered by endpoints of	must be covered by must be inside of
Line Geometry	end points must be covered by		
Polygon Geometry	must contain	must contain	must not overlap with

Regarding rules presented in Table 6-3 applied over line features:

- Dangles are objects with at least one end point that is not shared by another object. A dangle is often caused by inaccurate digitizing where an object extends beyond its intended intersection with a target object.
- A pseudo-node is an unnecessary node in a geometric link that is shared by only two objects. For example, a long link might be divided unnecessarily into many, smaller links by pseudo-nodes. Pseudo-nodes exist where two links meet; removing them creates a single polyline on one layer.

6.5. Map GIS Formats

Map GIS formats maintain the stylization used to display vector GIS and raster data on the actual map, with the reference to the actual data along with other map sheet information displayed outside the actual map area. Map GIS data formats are not frequently employed in surveying and mapping for road projects. Nevertheless, when deliverables include such files, the following points should be considered:

- Map features shall be displayed in the map section, while sheet outline and descriptions, such as, titles, scale, etc., shall be displayed in the layout.
- Symbolization provided for feature layers by MoMRAH shall be used when no basemaps are employed. When imagery is used as basemap, feature symbolization shall be adjusted according to the basemap's color scheme after consultation with the client.
- All external files used for map compilation shall be delivered additionally to the main file.

6.6. Other File Formats

Regarding all other file types included in the delivery of survey and mapping activities for road projects, the following rules are applied:

- In addition to the editable CADD/GIS Map files, every map produced shall be delivered in PDF file format with a resolution of at least 300 dpi.

- All reports shall be delivered in both editable formats (DOC/ODT/XLS etc.) as well as PDF format with a resolution of at least 300 dpi.
- Digital Terrain Models shall be delivered in ESRI TIN file format.
- Point Cloud Files generated by LiDAR or aerial Surveys, shall be preferably delivered in E57. LAS or LAZ formats can be also used.
- Field notes submission details and relevant file formats are covered in Section 4.4.6.

7. Horizontal Control (TPS)

Horizontal Control aims at establishing control points in the form of fixed points on the ground made by concrete or other solid materials at a depth that ensures stability. These points, also called monuments, have known coordinates in the given reference system, meaning that their locations in the datum are precisely defined, and serve as the basic reference for all surveying and map production works.

These points were historically connected to each other in the form of a network of triangles, so they are also known as "triangulation points". The accuracy (order) of these points varies depending on the purpose of their creation and according to the degree of accuracy required and the distance between the points.

There are multiple ways and methods through which these points are created, including Triangulation, Trilateration and Traversing. Among them, the traverse method is the simplest, easiest, and most common in the Kingdom in the past. The selection of the appropriate method depends on the nature and size of the project, the terrain of the site, and the accuracy and time required.

7.1. Establishing Control Points

Ground survey work depends on the established ground control points, installed in the vicinity of the area to be surveyed. When TPS is used, these points should form a closed traverse to facilitate site's control and verify observations with no less than five points, distributed in a fine geometric shape that allows each point to see the two neighboring points and to be close to the details to be surveyed. It is also desirable that the lengths of the traverse sides be approximately equal or identical, and its angles are not too acute nor too obtuse, rather than should be as close as possible to the above requirements.

All survey control points installed in nature should be referenced to the SANSRS. Therefore, an appropriate reference method should be chosen and implemented, including GNSS Surveys or the use of existing nearby control points established by any method. In the latter case, the following should be considered:

- Determining in advance the type, number, and accuracy of the points that will be used by obtaining their analytical description from the respective authority.
- Only existing control points of higher degree than the one that will be created can be used.
- When using existing control points, surveyors should properly maintain them and not expose them to any action that affects their stability and permanence; any discrepancy or difference in the values of their coordinates should be reported to the competent authorities.

7.2. Horizontal Control Points

Horizontal control points are a set of points on which at least two-dimensional coordinates are known (easting, northing, or longitude and latitude) with respect to the given datum. Since it is not feasible or economical for all the control points in the network to be at the highest possible accuracy, different levels of accuracy are used.

For horizontal control networks established with TPS, accuracy standards with respect to (a) distance and angular measurements, and (b) the order, are specified in Table 7-1. In lower orders, the specifications of Table 7-2 apply.

Table 7-1 Horizontal Control Accuracies

Order	Allowable Residual of Distance Measurement	Allowable Residual of Angular Measurement
First	1:120,000	2″
Second	1:60,000	4″
Third	1:30,000	5″

7.3. Instrument Calibration

TPS Surveying equipment should be calibrated and tested from time to time and adjusted if necessary. Requirements regarding instrument calibration posed in Section 4.2 for all topographic equipment, are also applied in TPS surveys.

In addition, all EDM devices shall be constantly checked, regularly maintained, and calibrated at least annually on an accurate baseline to determine the instrument constant and scale factor. EDM frequencies or vibrations shall be checked every 3 to 4 months.

Before using TPS it should be assured that all angles and bearings are observed and recorded in degrees, minutes and seconds, and all bearings must be reckoned and expressed clockwise from zero to 360 degrees.

7.4. Instrument Care

Surveying devices, especially TPS and TLS, are considered delicate sensitive devices in the sense that if they are exposed to the slightest shock, their work is severely affected, and they cannot be returned to their original condition except by referring to the specialized maintenance center. Therefore, surveying instruments should be used with care. Moreover, they should not be left unprotected especially when working on highways and areas with heavy traffic.

Surveying instruments should be avoided from falling on the ground and be protected from exposure to dust, extreme heat, and excessive humidity. Bellow, there is a list of subjects to be considered and avoided when using TPS:

1. Avoid straining the instrument, especially when it is lifted from its box. TPS should not be carried from the binoculars or the tribrach. Rather, the one hand should be placed under the base and supported by the other.

2. The instrument should be well connected to the tripod and be assured of it before leaving it.
3. Avoid attaching the instrument too tightly to the tripod, since this may cause damage to the fastening screws. Nevertheless, it should be well tied to it to ensure that the instrument does not slip out of the holder.
4. The instrument should be kept away from vibrations as much as possible, whether during use or not.
5. Do not use force in dealing with the instrument, as this may lead to the breaking of one of its axes.
6. When moving the device for a short distance, it should be carried vertically as much as possible, resting on the shoulder, with the horizontal movement screws being lightly tied so that it does not move while carrying it because it is easy to break the device if it hits anything and the screws are well tied. In the case of moving the device for a large distance, it is best to place it in its box.
7. A substance for absorbing moisture as well as a type of bactericide should be placed into the instrument box to protect from humidity, bacteria, and microbes.
8. An umbrella should be used during work to protect the instrument from the intense heat of the sun and rain, whether light or heavy, as exposure to heat caused by the sun leads to the expansion of the parts of the instrument and reduces its accuracy.
9. Instrument should remain stable during its operation. Tripod's legs should be secured in the ground, according to the nature of the land on which the instrument is used. If the ground is rocky, legs should be fixed through the cracks in the rocks. When placed on a tiled sidewalk, the tripod's legs should be fixed through the seams.
10. If the wind is severe enough to affect the stability of the instrument, the work should be stopped.
11. There is only one specific, designated position for the device to be successfully placed inside its box. Damage to the device can be caused by forcing or pressing it inside the box. These actions should be avoided because of the delicate and sensitive nature of such devices.

7.5. Instrument Setup

The means of data collection by the TPS requires an understanding of the basic principles of this device and familiarity with the process of initializing it, and identifying methods of measuring horizontal and vertical angles, as well as how to measure distances, and then how to make the necessary calculations and extract the coordinates for the observed points. The process of TPS initialization on a new station includes three stages:

Centering the instrument: It is the placement of the device so that the center or the extension of its vertical axis is exactly above the center of the mark of the respective station, and at the same time it is placed in an almost horizontal position, determined by the instrument's bubble. Several contemporary TPS instruments, use a laser plummet for the implementation of the vertical axis. In the other case the centering is carried out by means of optical placing through the optical plummet moving the tripod's legs with small movements until the intersection of the cross stitch in the optical plummet coincides with the center of the station point.

Horizontally adjusting the device: It is the process of putting the instrument in a completely horizontal position, by using the three leveling screws of the tribrach. The method is summarized in the following steps:

1. The rapid horizontal movement bolt is opened, and the device is turned until the instrument's longitudinal axis is parallel to the line connecting any two leveling tribrach's screws. Then these two screws are turned in opposite directions either inward or outward by the same amount until the longitudinal bubble is stable in the middle of its course.
2. The instrument is rotated by 90 ° so that its position is currently perpendicular to the previous position, and then the third tribrach's leveling screw is turned alone until the longitudinal bubble is stable in the middle of its course.
3. The previous two steps are repeated until the longitudinal bubble settles in the middle of its course, regardless of the direction that the instrument is turned. When the longitudinal bubble remains in the middle of its course at any position of the device, we have finished the process of adjusting the horizontality of the device.

Backsight setting: This is done by taking an observation on another point of known position with a target. Focus on the telescope should be adjusted and the "parallax" should be removed when directing it towards the target, in a way that the image of the target is clear and coincides with the cross stitch. The eyepiece should be adjusted so that its focus is at the level of the cross stitch as well. Any defect in obtaining the image applicable to the cross stitch will cause high systematic errors. By completing this process, the initialization process is over, and the instrument is ready for monitoring.

7.6. Observations

7.6.1. Angles

There are two ways to monitor the horizontal angles.

7.6.1.1. Individual angles method

In this method, each angle is monitored separately and independently of the other angle, and then the final value of each angle is deduced. To do so, two observations have to be made for each angle. This method is considered one of the most accurate methods for monitoring horizontal angles, but it has the disadvantage that it takes a long time in the monitoring process.

7.6.1.2. Cumulative method

This method is faster, but less accurate than the individual angles method because any error in one of the angles affects the next angle. This method is mainly based on the consideration that one of the directions is the basic observation direction and all other angles are calculated as observation differences from the basic observation. When the number of angles to be measured from a given station is high, it is preferable to use this method; since this is the actual case in surveying works, and it is considered the most common method in terms of employment.

7.6.1.3. Angle observations for control points

To create horizontal control points of the first, second and third order, the following instructions should be considered:

- Direct and reverse vertical angles shall be observed between nine in the morning and three in the evening.
- Horizontal angle should not be measured during the daylight between ten in the morning and two in the afternoon on clear and hot days, nor in the half hour following sunrise or before sunset.
- The angles on a full arc in the two telescope positions (direct and reverse) should be monitored to avoid the error that may occur because of the tilt of the vertical axis of the device.
- Angles should be monitored with different starting points on the horizontal disk of the device to reduce or limit the error in the horizontal circle of the instrument. The circle can be divided into two, three or four sections according to the number of arcs to be monitored each time. If only two arcs are required, one starts with zero and then 180 °. But if three are required, then one starts with zero, then 120 °, then 240 °. In the case of four arcs, one should start with zero, then 90 °, then 180 °, then 270 °, and so on.
- Specific and carefully lit objects must be used for both day and night angle monitoring.

7.6.2. Distances

Like any other measurement instrument EDMs employed in TPS (as well as any other EDM) are subject to errors which are usually caused by inaccuracies of the devices, inaccuracies in weather data, altitude differences, device, and target positioning, etc. These errors are usually determined by the manufacturers of surveying equipment as a constant value of Constant Part C_1 and a variable value of Variable Part C_2 given in relation to the actual measured distance. Therefore, the standard deviation S_d of the distance measurement d, is given by the following equation:

$$\boldsymbol{S_d} = \pm(\boldsymbol{C_1} + \boldsymbol{C_2} \times \boldsymbol{d}) \qquad \text{Equation 7-1}$$

Variable C_1 (constant) summarizes errors regarding inaccuracies of the devices, device, and target positioning, while C_2 (scale factor) includes errors resulting from weather conditions. C_2 is usually provided in *parts per million* (ppm). Weather condition errors dominate the total error when dealing with long-distance measurements, while positioning errors dominate the total error regarding short measurements. Unless expressly stated otherwise, the following requirements, specifications, and guidelines regarding distances shall be adhered to for all orders of horizontal control points:

- EDM measurements should be taken according to the procedures specified by the manufacturer. Both heights, regarding the instrument and reflector (target), should be measured within ± 5 mm and recorded with the measurement of distances.
- Distance measurements during the day between ten in the morning and two in the evening on clear and hot days should be avoided. The same also stands regarding the half hour following sunrise or before sunset, due to irregular changes in refraction.

- All EDM devices measure inclined distances directly, while other values such as horizontal and elevation differences are calculated by the measured distances and the vertical (zenithal) angles. To create horizontal control points with traverses, it is necessary to convert the slope distance first to the horizontal, then to sea level, and then to the reference to the datum’s ellipsoid. The horizontal distance is then computed according to the order of the survey: (a) first and second order surveys require obtaining control points elevations (obtained by differential leveling) as well as instrument and target height (b) third and lower order surveys may use the vertical angles as well as instrument and target height.
- Meteorological data, i.e., temperature, pressure, and humidity at the instrument’s position must be recorded in the field book. In first order traverses, angle monitoring, distance measurement, and weather data collection should be performed at a height of at least 1.5 m above the ground, to avoid unexpected differences in temperature.

7.7. Triangulation

Triangulation is the main method used in TPS control surveys, being highly effective, especially when surveying large areas. Before the introduction of GPS/GNSS technology, it was the main method for establishing control networks at country level. The method historically is based on creating a network of interconnected triangles, where all the interior angles of each triangle are measured directly on the ground, with the length of one side of one of the triangles being accurately measured and called the Base Line. Using the measured baseline and the observed angles, the lengths of the sides of all triangles can be computed.

Triangulation requires starting the survey from at least two points of known coordinates visible to each other to determine the azimuth and measure the baseline. The triangulation process also requires measuring all the angles of each triangle in the network, while in theory it is possible to measure only two angles and deduce the third angle mathematically; as such additional observations are introduced and the need for adjustment arises.

Similar to using angles, distance measurements can be used to define the geometry of a triangle, with a process called trilateration. Trilateration is the process of determining the location of a point by forming triangles to the point from known points involving distance measurements at known points. Modem TPS being able to measure horizontal angles and distances extend the notion of triangulation and trilateration combining them both with the method of least squares adjustment.

The significance of triangulations has been greatly degraded over the years that followed the introduction of GNSS methods. Nevertheless, is still an effective method, which should be employed in limited areas, instead of GNSS.

7.8. Traversing

Traverse is a method to establish control networks by placing survey stations along a path of travel, and then using the previously surveyed points as a base for observing the next point. Creating a traverse in nature should respect the following steps:

- Exploring and making a general sketch of the area.

- Selecting and marking of traverse points (stations) in nature.
- Make description cards for each station.
- Measure the traverse interior or exterior angles, as well as the lengths of the sides.
- Determine the azimuth of a traverse side.
- Calculate and adjust the data.

Table 7-2 and the instructions below provide the requirements and guidelines for creating horizontal control points with traverses. To achieve the standards and specifications specified in this table, sufficient specific procedures must be followed, whether they are related to engineering form, equipment and devices, or field procedures and data processing. Unless expressly stated otherwise, the following specifications and guidelines must be adhered to for all degrees of horizontal adjustment points using traverse techniques (MoMRAH, 2005):

1. Traverse sides are chosen so that they are close to the straight lines between the terminal stations, with a deflection angle not exceeding 50 degrees.
2. Traverse side lengths should always be close to each other within a factor of (1:3).
3. All station side lengths should be within a distance of D/4 where D is the distance between the first station and its orientation.
4. Cantilever or side shots are not allowed.
5. Every effort should be made to create intersecting traverses in order to strengthen the self-weakness of the traverse technique, to be used for least squares adjustment.
6. The orientation of at least one of the traverse's sides is set using already established control points.

Table 7-2 Requirements for Horizontal Adjustment with Traversing (MoMRAH, 2005)

Attributes and Features	**Order**	
	Fourth	**Fifth**
Relative Accuracy	1:10,000	1:5,000
Network Geometry		
The longest distance of a traverse between two given points (km)	20	10
Max number of sides	18	20
The least number of points to be focused on	2	2
Instrument Specifications		
Lowest theodolite reading (degree seconds)	3″	6″
Field work		
Directions		
Minimum number of shots	3	2

Attributes and Features	Order	
	Fourth	**Fifth**
Relative Accuracy	1:10,000	1:5,000
Maximum standard error (degree seconds)	2.5″	4″
Max difference from average (degree seconds)	6″	8″
Inverse vertical angles required for oblique distance reduction		
Number of Observations	2	2
Max difference between observations (degree seconds)	10″	10″
Maximum difference between observations (degree seconds): maximum line closing error (mm) l = average distance length; n = number of traverse sides	$50 \times l\sqrt{n}$	$50 \times l\sqrt{n}$
Distances		
Minimum accuracy for device	5 mm+ 5 ppm	5 mm + 5 ppm
Maximum difference between readings (mm)		
EDM	10	10
Minimum relative accuracy	1:60,000	1:30,000
Office work		
Max angular closing error (degree seconds) n = number of setups	$12 \times \sqrt{n}$	$24 \times \sqrt{n}$
Max positional closing error (meters) q = Distance (km)	$0.3 \times \sqrt{q}$	$0.54 \times \sqrt{q}$

7.9. Adjustment

All methods used for adjusting measurements, employ the notion of closure error, i.e., the difference between what is measured in the closure of a traverse and the actual point's known coordinates, a value that is used as a basis for tuning.

There are several steps to calculate the results for a traverse and adjust the angular closure error: calculate the direction in the starting position, get the values of the intermediate observed angles and calculate the closure error at the ending direction. This is followed by the correction phase, given that the closure error is within the permissible limits. The closure error is distributed equally to the traverse stations with a correction sign opposite to the one calculated. After correcting the angles their sum equals the theoretical sum of the geometric shape of the traverse. A similar methodology is followed regarding closure error in coordinates resulting in linear error distribution among the traverse sides.

When triangulation is used, the simple notion of the sum of angles of a triangle is employed to adjust the measurements. The plane triangle is characterized by the well-known geometric relationship, which is that the sum of its interior angles is 180 degrees. This relationship is used to adjust the triangle, and there are two cases:

- If the observations are of equal weights; in this case the triangle is adjusted (i.e., finding the most likely value of its interior angles) by distributing the closure error evenly over the interior angles. So, their sum is exactly 180 degrees.
- Given that the observations have different weights, the triangle is adjusted by correcting its observed angles by an amount inversely proportional to the weight of the angle. This is the actual case when angle observations have been repeated multiple, but different times on each triangle corner, or when different instruments are used for each observation.

It should be also noted here that in the case of the right triangle, the surface of the Earth changes the shape of the triangle, and the sum of its internal angles becomes a little more than 180 °. This is called the "spherical increase", which is a very slight increase, about a second for a triangle of about 125 km^2. Therefore, given that TPS control surveying for road projects only applies to limited areas, most of the triangles monitored by surveyors are considered flat and the spherical increase in them can be neglected.

All methods used in horizontal control, triangulation, trilateration, traverses can be computed using least squares adjustment. Least squares, is the best adjustment method, because it simultaneously considers all factors affecting measurements and limits them, resulting in the most probable value, being also consistent with probability theory. Moreover, when multiple traverses interconnect to form a network, points common to two or more traverses need to be adjusted considering both traverses, something that can't be done with traditional traverse computation. Finally, previously described simplistic methods for adjusting traverses and triangle observations, are essentially a simple form of least squares adjustment.

Modern TPS and relative surveying software provide the ability to adjust TPS data to provide final point locations, regardless of the network shape formed by the actual observations, using least squares.

The following rules apply for horizontal control adjustment with TPS:

- Least squares adjustment shall be used for adjusting horizontal control networks.
- Weights of distance measures shall be computed according to the values of standard deviation given by the formula provided in Section 7.6.2.
- Weights regarding angles/directions observations shall be computed according to the values of the horizontal angle precision provided for each TPS according to its vendor.
- Chi-squared test used to determine whether there is a statistically significant difference between the expected deviations and the actual observations shall be performed for accepting adjustment results.

8. Vertical Control (Leveling)

8.1. Vertical Control Points

Vertical control refers to procedures that establish the precise values of elevation or heights in vertical control points, usually referred to as benchmarks. Benchmarks' elevations are provided relative to a datum, usually mean sea level, which is also expressed by the Geoid model. Vertical control points are placed in close positions to one another, typically along railroads, highways etc. Vertical datum in KSA is implemented by KSA-VRF14 and the respective geoid model KSA-GEOID21.

Vertical control is established with leveling. The scope of leveling is to measure or verify the height of specified points relative to a datum. It is commonly used in geodesy and cartography to measure geodetic height, and in construction to measure height differences of construction features. It is also known as direct of differential leveling and measures vertical distances directly on a graduated rod, or leveling staff, with the use of a leveling instrument such as a dumpy level, transit, or theodolites. Different lines of sight are established during differential leveling. A known benchmark elevation and an unidentified point with an unknown elevation are the spots of the two readings along the line of sight. To get the elevation of the unknown point, a subtraction of these two readings establishes the unknown elevation difference.

Among the sites suitable for creating benchmarks are pavilions, walls of ferries, prominent bridges, columns buildings entrances and boundary walls. Benchmarks, have flat surface so that the base of leveling staff does not change when it is directed or turned.

The difference in elevation between two points can also be determined trigonometrically using vertical (zenithal) angles and horizontal or inclined distances with a method called trigonometric heighting; however, this method is unsuitable for high order control surveys.

Differential leveling shall be used for first, second order accuracy using appropriate survey equipment and practices. Heights for third order shall be provided by differential leveling and trigonometric heighting. Fourth and lower order accuracy shall be provided by differential leveling, trigonometric heighting, or with GNSS methods (Chapters 10 to 12).

8.2. Equipment

Differential leveling is performed with level instruments, an optical instrument that is used to establish or confirm points in the same horizontal plane. It is also used to determine the relative heights of objects or markers when used in conjunction with a leveling staff. It is often used in building and surveying to transfer, measure, and set heights of well-known features or markings.

Accuracy of levels employed in control surveys, regardless of the order of the survey, shall provide accuracy of at least 1.5 mm per km of double leveling. Trigonometric heighting shall be performed with typical TPS.

Leveling shall be performed using precision automatic levels and accurate level staffs graded to 1 cm or smaller. Digital electronic levels and appropriate bar-coded staffs should be employed to increase accuracy and help detect and avoid reading and registration errors. In the case normal (non-digital) levels are employed, special additional requirements are applied as described in the next sections.

8.2.1. Instrument Calibration

Leveling devices should be calibrated and tested from time to time and adjusted if necessary. In addition to the general requirements regarding instrument calibration of Section 4.2 leveling instrument shall adhere to the following:

1. The collimation error in all leveling devices shall be checked before field monitoring, and it should be controlled whenever necessary.
2. The so-called “Peg Test” should be performed at least once a week and be used to correct the collected data according to the test results if necessary.

8.2.2. Instrument Care

As levels are delicate and sensitive instruments, the rules regarding TPS instrument care of Section 7.4 also apply here.

8.3. Differential Leveling

Most of the vertical control points are established using differential leveling where height differences are measured in small parts, connected by leveling traverses. Parts are then added together to determine the total height differences between the concerned points. Table 8-1 sets the requirements for vertical control points established by differential leveling method. In addition, the following procedures should be taken into consideration (MoMRAH, 2005):

1. Leveling traverses when creating points shall start and end at points of the same or higher degree of accuracy.
2. Backsight and foresight distances should be balanced. Placing the device midway between the front and back reading points helps eliminate the error arising from the refraction of rays (due to passing through layers of air of different densities), the error arising from the curvature of the Earth's surface and the error from the collimation between the horizontal bubble and the true instrument’s horizontal axis.
3. Backsight, foresight, and leveling instrument positions should be in near-colinear locations.
4. Leveling observations should be performed between nine in the morning and three in the evening.
5. Level instrument during the leveling process must be placed on a solid base or ground. In soft areas, leveling staffs with nails must be used whenever possible. In the case of sandy soils, wooden staffs with a nail to be used as a turning point should be employed. As for concrete or solid surfaces, a flat solid base should be used.
6. Points, whose heights are measured at the end of the daily work, shall be well defined and on fixed locations, as they are used when completing the work at any later time.
7. Newly established benchmarks must be prepared several weeks before the first leveling of the line, to give it enough time for stability, consolidation, and stability.

Table 8-1 Vertical Control (Leveling) Requirements (MoMRAH, 2005)

Attributes	**Order**				
	1st	**2nd**	**3rd**	**4th**	**5th**
Precision. Max closure per line (mm) *L* = traverse horizontal length (km)	$4\sqrt{L}$	$8\sqrt{L}$	$12\sqrt{L}$	$20\sqrt{L}$	$50\sqrt{L}$
Network Geometry					
Maximum independent length between benchmarks (km)	300	100	50	40	30
Maximum interval between benchmarks (km)	6	5	4	3	2
Longest distance per part (km)	2	2	2	2	2
Minimum number of benchmarks of equal or higher order to bind to	3	2	2	2	2
Instrument Calibration					
The type of scale used: (IDS) Invar Double Segmented; (ISS) Invar Single-Segmentation	IDS	IDS	IDS	IDS / ISS	ISS Wooden Rods
Smallest height reading (mm)	0.10	0.10	0.10	0.10	0.10
Maximum line-of-sight error per shot (mm/m)	0.05	0.05	0.08	0.10	0.15
Field Procedures					
Minimal monitoring method; D = Digital level M = micrometer; W-3 = three capillaries;	D	D	D	M / W-3	M / W-3
Working tracks DR = Dual SR = Single SRDS = Double and single at the same time	DR/ SRDR	DR/ SRDR	DR/ SRDR	DR/ SRDR	SR/ DR
Longest shooting distance (meters)	50	70	80	90	100
The largest difference in the distance between the forward and backward readings for each position (meters)	2	4	6	8	10

Attributes	Order				
	1st	2nd	3rd	4th	5th
Minimum height of line of sight from the ground (m)	0.5	0.5	0.5	0.5	0.5
The largest difference between the upper capillary and the lower capillary in the reading of the three capillaries (mm)	-	-	-	2	3
Maximum difference between highest and lowest height difference measurement per post (mm)	0.3	0.4	0.7		
Office Work					
Closure between back and forth (mm) *L* = traverse horizontal length (km)	$3\sqrt{L}$	$6\sqrt{L}$	$9\sqrt{L}$	$16\sqrt{L}$	$35\sqrt{L}$
Maximum closure error per traverse or loop (mm) *L* = traverse horizontal length or total loop length (km)	$4\sqrt{L}$	$8\sqrt{L}$	$12\sqrt{L}$	$20\sqrt{L}$	$50\sqrt{L}$

Some general observations about leveling work can be summarized in the following points:

1. The leveling process must always begin and end with a reading on a fixed level mark (benchmark).
2. The correctness of the leveling staff shall be checked before use, especially near the joints in the case of multi-piece staff. Also, the leveling staff should be checked whether it is inverted or straight.
3. Leveling device and the respective leveling bubble should be checked and adjusted before starting the height measurement process.
4. Leveling bubble should be in its middle when observing the height or taking the reading at every point included in the leveling process. Therefore, leveling bubble placement check should be performed before and after a reading is taken.
5. The distance between level device and level staff should not exceed 100 and 50 m, in normal and accurate leveling works, respectively (see also Table 8-1).
6. The leveling staff shall be vertically fixed; the image of the capillaries shall be applied to the image of the staff completely so that there is no vibration in the image of the capillaries when moving the eye.
7. Transfer points should be chosen to be in flat, stable, and solid locations. The position of the leveling staff at the turning points must be fixed so that it does not decrease or rise when it is rotated to face the device in its new location.
8. Lens should not face the sun, as it is difficult to monitor the readings.
9. Device should not be moved until after taking a forward reading (at a turning point or the end point of the project); surveyor's body should not be moved until after taking a back reading (for a fixed level mark or a turning point).

10. In times of extreme heat, an umbrella shall be placed over the device to protect it from the heat. Under such circumstances readings on the leveling staff shall not be taken over long distances and should not be greater than 0.5 m to reduce the effect of ray refraction. The heat of the sun also affects the texture gradations of the rod, the leveling bubble, and the curvature of the surface of the leveling tube covering and containing the bubble.
11. To avoid the accumulation of water droplets on the objective when it rains, it is preferable not to do the leveling work under such circumstances. If it is necessary to work in rainy weather, the lenses should be protected with barriers or other protective tools and an umbrella shall be placed over the device to achieve clarity in the monitoring process.
12. Sketch showing the distribution mark must be clear and accurate.
13. The following tools should be used besides the leveling device and leveling staff: tape, poles, umbrellas, nails, pegs, short iron rods, iron angles, hammers for hammering, paint, colored chalk, simple drawing tools, pens, an electronic calculator.
14. Calculations in normal leveling works and limited distances, need not consider the curvature of the earth and ray refraction due to the rather small effect in the final result.
15. Some leveling devices are equipped with a circle to measure the horizontal angles which helps to determine the locations of level points. Moreover, stadia capillaries, stand for measuring distances (by multiplying the difference between the readings of the upper and lower stadia capillaries by a specific constant that follows the type of device). Given this information, and known point and backsight coordinates, the point on which the staff is placed can be determined. However, the accuracy is rather low when compared to the accuracy of TPS; therefore, such use should be avoided.

As already stated, electronic (digital) levels should be preferred for high accuracy leveling works. Nevertheless, when simple non digital levels are used, the following instructions shall be also followed:

1. A notebook suitable for leveling work shall be used to record point numbers and various readings, e.g., heights and horizontal distances. This notebook facilitates the writing process and helps to avoid clerical errors to a large extent. It is also considered a record that can be kept in the future for audit purposes and for the use of field notes.
2. The fixed level signs shall be ensured of being correct.
3. Care should be given on writing the different readings in their correct places and ensuring their correctness.
4. The first reading that should be entered into the leveling table is a back reading and the last reading is a front reading.
5. Rod readings should be done using one of the two alternative ways: (a) Reading the average middle capillary, and (b) Reading the average of the three capillaries.
6. Most leveling devices contain, one basic horizontal capillary and two additional horizontal capillaries (stadia capillaries), provided to estimate the distance between the device and the staff. Therefore, attention should be given to choosing the middle capillary.
7. If the user is not experienced, it is advised to take the readings in all three capillaries. As a check, the difference between the upper and middle readings should be equal to the difference between the middle and lower readings.

8.4. High Accuracy Leveling

In leveling works that require high accuracy (first and second order), the following additional instructions and requirements shall be considered (MoMRAH, 2005):

1. Only electronic digital levels shall be employed.
2. The leveling device shall be daily adjusted.
3. Least squares adjustment shall be used to distribute the permissible errors.
4. The bubble used in monitoring and its gradations should be checked.
5. The bubble shall be continuously monitored when readings are taken.
6. The errors caused by earth curvature and ray refraction shall be considered for all calculations.
7. Distance between the leveling device and the staff shall not exceed 50 m.
8. The leveling device should be protected from the effects of the sun and wind by using an umbrella and avoid, as much as possible, taking a reading of less than half a meter on the height to avoid the effect of the glare of the ground and ray refraction.
9. Leveling staff shall always be fixed on a steel triangular base or any other suitable flat solid base.
10. Good-quality leveling staff that resists changes in temperature and humidity, should be used.
11. It is not recommended that accurate leveling works should be carried out on days of strong winds, as these winds cause vibrations in the body and the equipment. In emergency cases, it is recommended to protect the device and use a short scale as well as take readings from short distances.

8.5. Adjusting Vertical Data

Leveling work is characterized by the fact that the effect of most systematic and random errors is directly proportional to the length of the leveling line. For this reason, systematic errors in leveling are avoided by placing the leveling device halfway between the two points (making thus the front reading distance equal to the back reading distance). On the other hand, random errors in leveling, are avoided by adjusting the measured heights: starting the leveling from a benchmark of known height, measuring the height differences and close to another or the same benchmark, then, the so-called leveling loop is completed. By closing the loop, the difference, i.e., closure error, between what is calculated in the closure and the actual known benchmark height can be determined. This closure error is the basis for adjusting the leveling traverse. If this error is within the permissible error range (Table 8-1), the leveling traverse has to be adjusted.

The single adjustment line is adjusted by distributing the closure error either on the differences in the observed heights (in proportion to the length of each side to the sum of the sides in the line), or on the heights calculated for each point (in a manner proportional to the cumulative distance of the point). To facilitate the calculation, we find the so-called *correction factor* (m), by dividing the amount of the closure error in the opposite of its algebraic sign by the total distance of the line.

$$\boldsymbol{Correction\ factor\ (m) = -\frac{Closure\ Error}{Total\ Distance}} \quad \text{Equation 8-1}$$

Then the height difference of each point is adjusted by multiplying the correction factor with the partial distance of this point. Also note that the correction factor is positive if the closure error is negative, though it turns to negative if the error is positive.

While this simplistic approach can be applied in single leveling traverses, overlapping traverses with common intersection points, require more complex calculations. In this case, leveling traverses shall be adjusted simultaneously using least squares adjustment with the help of relative weights for each leveling side, which are determined inversely proportional to the side length.

The effects of the levelling instrument collimation error should be minimized by field procedures demonstrated in previous sections, by balancing foresight and backsight lengths, as well as using well balanced instruments (NOAA, NGS, 1982). Nevertheless, when instrument collimation error is not zero, a correction is added with the resultant algebraic sign to the observed elevation difference using the following formula:

$$\boldsymbol{C_c} = -(\boldsymbol{e} \times \boldsymbol{SDS})$$ Equation 8-2

where C_c is the level collimation correction in millimeters, *e* is the collimation error factor in mm/m and *SDS* is the accumulated difference in sight lengths for the section expressed meters.

To minimize the effect of refraction, corrections to observed elevation differences should be computed, following the methodology provided in (NOAA, NGS, 1982), or any other relative, up - to - date approach.

Astronomic corrections to account for the effect of tidal accelerations due to the Moon and Sun on Earth's equipotential surfaces should not to be taken into account for road survey purposes due to their limited effect, i.e., 0.1 mm/km at most (NOAA, NGS, 1982). The same also stands regarding orthometric (gravity effect) due to their limited effect in road projects.

The following rules apply for vertical control adjustment leveling:

- Least squares adjustment shall be used for adjusting vertical control networks. All above corrections (collimation, refraction etc.) should be applied into elevation differences before applying least squares adjustment.
- Weights of height differences shall be computed inversely proportional to the total side length.
- Chi-squared test used to determine whether there is a statistically significant difference between the expected deviations and the actual observations shall be performed for accepting adjustment results.

It should be noted here that all above processing and adjustment procedures regard vertical control for the purpose of road surveys, being different, and less demanding, than the procedures applied by GASGI at the national level, according to the respective NGS standards.

9. Topographic Survey (TPS)

Topographic surveys aim to gather planimetric and height survey data about the natural and manmade features, for the purpose of preparing topographic maps, as well as Terrain data being used for DTM. Topographic surveys are commonly supported by horizontal and vertical control of third and lower order accuracies.

Traditional fieldwork in a topographic survey with TPS takes place by initially creating a basic framework of local horizontal and vertical control points, i.e., stations. Once these points have been established, surveyors locate, measure and document in field notes, the recognized feature details in the vicinity of each station. These details consist of all natural or manmade features that appear on the map as described in Sections 5.2 and 5.3.

In recent years, extensive use of TPS tends to be replaced by GNSS methods, i.e., RTK and PPK, since they require smaller crews, and are not dependent on the rather small vicinity of each station to measure feature details. Nevertheless, TPS methods are still preferred in cases of high vegetation and urban environments where GNSS methods are limited by the available sky visibility, or when relative precision required by the survey cannot be met by GNSS techniques.

9.1. Locating Stations

The location of stations is determined based on the nature of the project along with the features of the surveyed area and the type of work required to complete the project. The coordinates of a station can be known in case of the establishment of a previous survey that took place in the past; this is called "Operating on a known station". On the other hand, when there is no previous established station of known coordinates, then the need to establish a new one arises. Additionally, new stations can be established to meet the need for surveying more information and details of the environment and to expand the network of the control points. The coordinates of the new station can be determined by the following methods:

- Free Stationing: Free stationing or resection, is a method of determining a location of one unknown point in relation to known points. Using this method, TPS can be freely positioned so that all survey points are at a suitable sight from the instrument. Then bearings and distances are measured to at least two known points of a control network. Modern TPS support this method and directly determine new station's coordinates given measurements and used control point coordinates. Free stationing is the method that should be preferred to establish new stations supporting TPS topographic surveys.
- Traverse Method: This is the same as the traverse method used in horizontal control. When used for locating stations to support topographic survey, cantilever or side shots are allowed. This method should be applied in the absence of a suitable number of control points in the vicinity of the newly established station.
- GNSS RTK: This method should only be used when conditions of accuracy and precision permit its use: when no better horizontal accuracy of 30 mm and vertical accuracy of 40 mm is required.

9.2. Ground Survey Methods (TPS)

As mentioned, the relationship that needs to be determined between features and details of artificial or natural environment, to represent such features on maps, is expressed by measuring distances, angles etc. The methods that use such procedures as measuring distances and angles are called Ground Survey Methods (MoMRAH, 2005). These methods can be categorized into seven distinct categories based on the procedure of measurements and observations. These categories are:

- Intersection of points by distance measurements.
- Extension.
- Orthogonal.
- Traverse.
- Polar.
- Intersection of points by angle measurements.
- Resection.

For a better understanding of the methods, auxiliary figures are used. The figures represent the foundation of a building described by vertices A, B, C, D, and a control network of known coordinates (vertices 1,2,3,4) around the building of interest.

9.2.1. Intersection of Points by Distance Measurements Method

This method is one of the simplest and easiest to perform. It is mainly used in construction sites to assist in the process of raising buildings and constructions by defining their boundaries. As shown in Figure 9-1 the distance between the points with known positions (1 and 2), and the corners of foundation A and B of the building are measured. The distances are measured using a measuring tape, an EDM or a surveying device. Then the coordinates of points A and B can be calculated. Subsequently the coordinates of points C and D can be related to the coordinates of points A and B in the same manner. Coordinates can be calculated by using CADD software or other related software.

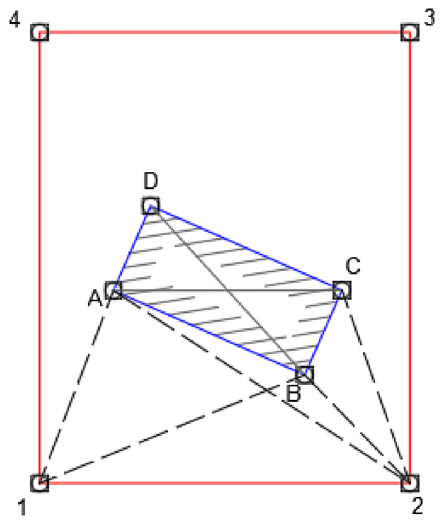


Figure 9-1 Intersection of Points by Distance Measurements Method

9.2.2. Extension Method

This method is mainly used in construction to determine boundaries of single standing buildings. The extension of the direction of the building to the lines connecting the control points are used to determine the position of the boundaries of the building. In Figure 9-2 the lines A-B and C-D are extended to the lines connecting the control points 1 - 4 and 2 - 3. The distance between their respective intersection of each vertex of the building is measured.

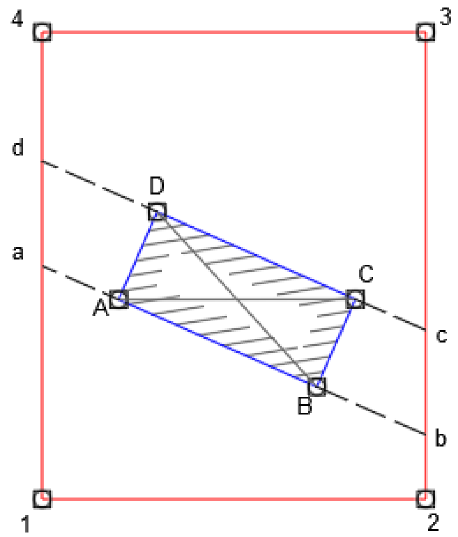


Figure 9-2 Extension Method

9.2.3. Orthogonal Method

This is a method that was used mainly in the past by measuring points using auxiliary traverse and survey lines to determine the coordinates of points of interest. At the present this method is considered redundant and ineffective because of the development of TPS (see Figure 9-3).

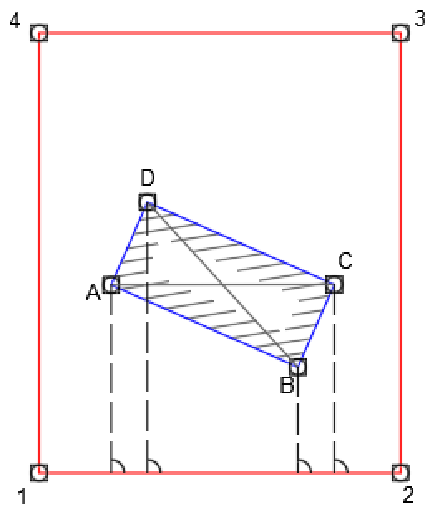


Figure 9-3 Orthogonal Method

9.2.4. Traverse Method

The most used method applied in surveying procedures nowadays fully takes advantage of TPS features. Placing of TPS along a line of travel, using the previously surveyed point as a base for observing the following point, is known as a traverse network. The basic purpose of the traverse method is to create additional control points. It is frequently required in urban environments, for example, to survey backyards with limited access and visibility. As shown in Figure 9-4 points A through D can be related to the control points of known position, 1 and 4 by measuring the deviation from the direction 1 - 4 to the point A, using TPS which can also calculate its coordinates.

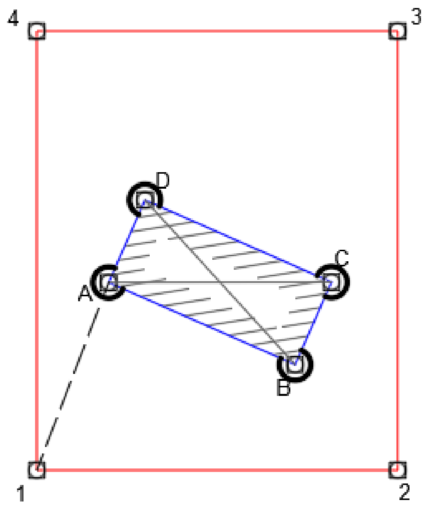


Figure 9-4 Traverse Method

9.2.5. Polar Method

Polar method finds its appliance in crowded urban areas and low visibility conditions, mostly for surveying high buildings. Polar method determines the direction and distance of each new point (A through D) from a station of known coordinates. The method is shown in Figure 9-5 where 1 is assumed to be the station and 1 - 4 the referenced direction. The polar technique identifies several points from a single station, in contrast to the traverse method, which uses each new point as the station for the following point. It is advised for some of the points surveyed to be determined again using different station and referenced direction (for instance 4 as the station and 4 -1 the referenced direction). All necessary calculations can be performed by TPS instantly, on site.

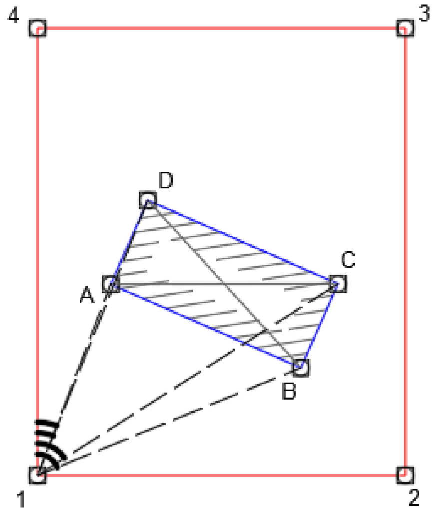


Figure 9-5 Polar Method

9.2.6. Intersection of Points by Angle Measurements Method

This method is mainly used in hard-to-reach locations of low visibility and unpreferable conditions. In this method the coordinates of the surveyed points (A through D) can be calculated by measuring their directions from several different stations of known coordinates. The points should be observed by at least two stations that share mutual visibility between them and the points surveyed. In Figure 9-6 the known stations are assumed to be 1 and 2 and the points A through D, are observed by measuring the deflection angles between the points surveyed and the stations. The intersection of two rays from two known points can be used to get any point's coordinates by solving a triangle whose base and two angles are known. In recent years with the aim of TPS and reflector less EDMs the significance of this method has been degraded, since coordinates of unreachable points can be directly calculated with the polar method and a reflector less distance measurement.

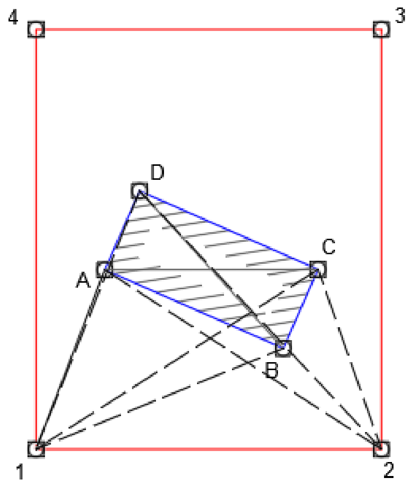


Figure 9-6 Intersection of Points by Angle Measurements Method

9.2.7. Resection Method

Resection method is a technique of determining the position of one unknown point in relation to other known points. Although the resection method is mathematically complicated, it has the benefit of concentrating the work in one place. Its steps can be summed up as focusing on the point on which a TPS is placed (A, B, C, or D), followed by relating it to at least three points with known coordinates (1, 2, 4) and measuring angles, though it is preferable to monitor a fourth point for verification (Figure 9-7). In recent years with the aim of TPS and reflector less EDMs resection method has been enhanced, forming the notion of “free station” (Section 9.1).

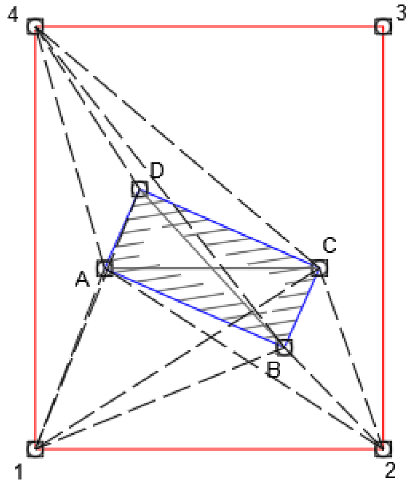


Figure 9-7 Resection Method

10. GNSS Surveying

Global Navigation Satellite System (GNSS) refers to several constellations of satellites broadcasting signals from space that transmit positioning and timing data to GNSS receivers. The receivers then use this data to determine location. Generally, GNSS provides global coverage. GNSS, has been introduced as a term that generalizes the notion of GPS, which was the first actual operating satellite constellation serving for this purpose. GNSS includes Europe's Galileo, the USA's NAVSTAR Global Positioning System (GPS), Russia's Global'naya Navigatsionnaya Sputnikovaya Sistema (GLONASS) and China's BeiDou Navigation Satellite System.

To define a precise horizontal location on earth, signals from at least three satellites are required. For the determination of vertical position at least four satellite signals are needed. Due to its positioning capability, GNSS is widely used for surveying. A single GNSS receiver observing sessions of only a few hours can yield three-dimensional positions with accuracies of a few centimeters. Differential techniques evolved soon after the introduction of the first constellation (GPS), allow to accurately and, in some cases, instantly, define the position of a point on the surface of the earth.

GNSS survey guidelines continually evolve with advancements in equipment and techniques. Changes to these guidelines are expected as these advancements occur. Recent changes include the adoption of IMU/inclinometers on GNSS receivers allowing thus to get tilt measurements and determine point positions regardless of the slope of the pole on which the receiver is mounted.

10.1. GNSS Principles

10.1.1. System Definition

GNSS refers to systems supported by several man-made constellations of satellites that allow determining locations, speed, and direction all over the world by land, sea, and air, 24 hours a day, and under any weather condition. Each system/constellation may be used on its own or together with other systems, depending on the employed receiver, for navigation and survey purposes, providing great capabilities in this field with high accuracy and speed.

10.1.2. GNSS Mechanism Principle

The idea of a space system for navigation depends, in a simplified way, on measuring the time required for electromagnetic waves to travel the distance between the satellites that transmit these waves from space and the receiver that receives them on the surface of the Earth. Using the known value of the speed-of-light, the distance between the transmitting satellites and the receiver on the Earth's surface can be determined. Then, by collecting and processing raw data from several satellites, for at least an hour, the coordinates of the ground point can be obtained with an accuracy of 0.01 ppm (MoMRAH, 2005).

10.1.3. GNSS Components

All GNSS systems consist of three main segments: the space, the control, and the user's segment, each one described in the following sections (Figure 10-1).

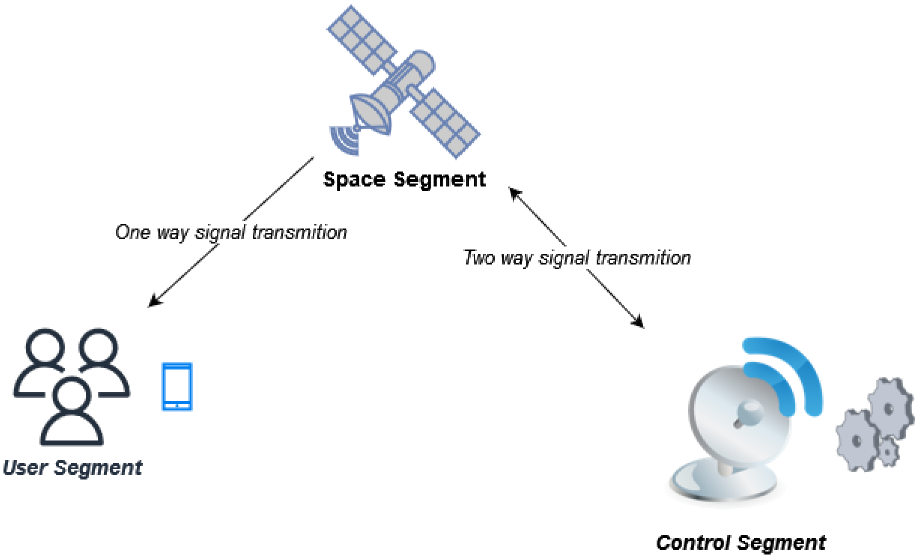


Figure 10-1 GNSS Components

10.1.3.1. Space segment

The emergence of the space segment for USA's NAVSTAR GPS, began in the mid-1970s with a group of satellites developed by the US Department of Defense revolving around the Earth in a polar orbit at the equator, at an angle of 55 degrees and at an altitude of approximately 20,200 km from the surface of the Earth. These satellites are distributed regularly in six orbits, with three satellites in each orbit, in addition to the presence of other backup satellites. Each satellite revolves around the Earth about two complete revolutions per day.

The development of GPS was followed by the Soviet GLONASS constellation, European Galileo, and China's BeiDou. Galileo satellites orbit at an altitude of 24,000 km. Galileo represents the first global system for determining the location for civil use owned by a group of countries, operating on a commercial basis. The multiplicity of these constellations of satellites indicates the importance of this technology, as it achieves its permanence and keeps it away from monopoly and control. Satellites carry ultra-accurate atomic clocks allowing them to precisely determine time information.

The satellite missions are as follows:

- Modify its orbit according to the directions sent to it by the control unit.
- Receive and store information sent from the control unit.
- Get the exact timing from the atomic clocks installed on it.
- Send information to users by specific frequencies and codes.

10.1.3.2. Control segment

The mission of control segment is to track and monitor the satellites in their orbits in space, perform calculations to update the navigational information they transmit, as well as perform all relevant operations necessary to control these satellites on an ongoing basis. Regarding USA's GPS Control Unit, it consists of the following:

- Five monitoring stations in remote locations around the world.
- A main control station.
- Three stations to send control signals and navigational information to the satellites in their orbits.

Monitoring stations' locations have been surveyed with very high accuracy. They employ precise receivers to collect satellite orbit data and send collected information to the main control station. Main control station processes data collected by all satellites determining their locations, speeds, and time differences with high accuracy. Then the main station uses these results to update the navigational information for each satellite and make the necessary corrections, and then sends it to the satellite in question via one of the three transmitting stations. This way each satellite is provided with new navigational information and correct timing data at least once a day to ensure the accuracy of the system.

10.1.3.3. User segment

The user segment consists of the receiver that each system's user employed to track its locations, speed, and navigation data, according to his needs. The aim of these devices is to receive the signals transmitted by the space segment using an appropriate antenna and process them to perform the necessary calculations and get the respective results; receivers according to their scope of use may also be equipped with a display screen, control units, or other transmitting devices e.g., Bluetooth, Wi-Fi etc. Receivers perform all necessary calculations for converting the signals emitted by the satellites into antenna location coordinates (longitude, latitude, and altitude).

10.1.4. GNSS Features and Application Areas

GNSS surveying shows several advantages regarding traditional geodetic surveying methods, as follows:

1. There is no need for a mutual view between the observed points.
2. Control point establishment can be achieved with a small fraction of the respective cost and time required with traditional methods.
3. Accurate results can be obtained very quickly.
4. Equipment is easy to use; thus, the need for specialized and trained personnel degrades.

GNSS are employed in several aspects of surveying works, from control and location surveys to bathymetric and mobile mapping surveys. Here are some of these applications:

1. Control surveys.
2. Surveying works, producing, and updating maps.
3. Marine and bathymetric surveying works.

4. Control surveys for photogrammetry
5. Construction surveys, plot corners staking etc.
6. Fleet management - accurate navigation and trajectory determination, whether by land, sea, or air.
7. Tracking the movement of targets (missiles, aircrafts, ships, satellites etc.).
8. Studying the movement of the earth's crust.
9. Determining the magnitude of Earth's gravity.

10.2.GNSS Survey Techniques

Each GNSS satellite carries very accurate clocks that are used as a reference for timing and frequencies in the GNSS systems. GPS satellites transmit unique identifying information about their location, timing and navigation based on two frequencies: the L1 frequency of 1,575 MHz (equivalent to a wavelength of about 190 mm) containing a precise code P, and a general code C/A, and the L2 frequency of 1,227 MHz (equivalent to a wave of about 244 mm in length) containing only a minute code P that has a wavelength of about 29.3 m compared to the C/A code with a wavelength of 293 m. Other GNSS satellites transmit similar information on other frequencies.

Among the messages broadcasted by satellites, their navigational properties are transmitted in the forms of ephemeris and almanac. Ephemeris contains information on week number, satellite accuracy and health, age of data, satellite clock correction coefficients and orbital parameters valid two hours before and two hours after the ephemeris time. Almanac contains less accurate orbital information than ephemerides valid for a period of up to 90 days.

All GNSS surveying methods are based on the application of the relationship between distance, speed and time:

$$\boldsymbol{distance} = \boldsymbol{time} \times \boldsymbol{speed}$$ Equation 10-1

The receiver measures the time required for the arrival of the satellite waves - each satellite separately - and the previous relationship is applied, so that the distance between the receiver and the satellite can be obtained. By knowing the orbital position of the satellite at the moment of observation, the theoretical distance of the location of the receiving point in relation to this satellite are determined.

GNSS observations are processed to obtain exact point locations provided by their geocentric coordinates measured in relation to the center of the Earth (X, Y, Y) according to WGS 84. Since there are three unknown coordinate elements (X, Y, Z), at least three satellites have to be observed to solve the three unknown coordinate elements, in addition to monitoring a fourth satellite to cover the lack of very accurate clocks in the receiving part, resulting in incorrect timing. The more satellites there are, least squares adjustment is applied to obtain more accurate results.

In general, there are three basic GNSS techniques:

- The absolute determination of the point.
- Differential GNSS (DGNSS).

- Relative determination (using carrier phase observations).

GNSS usage in surveying requires the presence of two or more GNSS receivers, obtaining simultaneously carrier phase wave observations. Relative positioning using L1 and L2 frequencies provides very accurate results, since the signals are simultaneously received by two or more receivers of the same group of satellites. These observations are processed to identify baseline components between observation points, providing results with an accuracy of 0.01 parts per million. A GNSS baseline is created between two points that are simultaneously occupied by GNSS receivers. Through baseline post-processing, a vector establishing the relative 3D coordinate is possible and the differences between both points is computed.

Accuracy in GNSS surveying depends on many factors, such as:

- The number and locations of the simultaneously observed satellites.
- Duration of the surveying.
- The location of the network and the length of the baseline.
- Astronomical calendar, weather effects, models and surveying methods.

Among the elements mentioned above, clock and atmospheric errors (from the upper and lower atmosphere) play a crucial role in the accuracy of the observations. When using differential data processing, e.g., static, PPK, RTK etc., the effects of clock errors can be eliminated or minimized to a certain extent. Unfortunately, this does not apply in the case of atmospheric refraction errors, especially when long baselines are used, where the errors of the upper layers of the atmosphere vary according to the latitude (they increase as the latitude increases) and according to the season and the timing of surveying during the hours of the day. The value of the refraction error in the upper atmosphere ranges from 1 to 10 parts per million. Dual frequency receivers can solve delay problems while such errors can be equalized or reduced using differential measurements.

GNSS techniques being currently in use for surveying purposes include:

- Fast Static.
- Static.
- Continuous.
- Real time Kinematic (RTK).
- Post processing Kinematic (PPK).
- DGPS.
- Precise Point Positioning (PPS).

Before choosing the appropriate technique, the following factors should be considered:

- Purpose of the collected data.
- Horizontal and vertical accuracy requirements of the project.
- Local and international best practice.
- Local survey requirements.
- Cost of equipment.
- Timely solution.

Based on the answers to these questions, one may choose among the previous techniques. In the following paragraphs all currently available techniques are discussed, while Table 10-1 summarizes this discussion.

All of the above methods used in GNSS surveys depend on the proximity and availability of CORS stations, or another receiver also mounted over an existing control point (called a reference point), receiving observations during the same time period; receivers mounted on these reference points are called bases, while receivers used to measure new points are called rovers.

Table 10-1 Choosing an Appropriate GNSS Surveying Type - Advantages and Disadvantages

Survey type	Advantages	Disadvantages
Real time Kinematic (RTK)	Real-time corrected positions in a known coordinate plane. Able to navigate to and compute geometries of data points in the field. Provides final positions, no need for additional calculations, minimizes resources and logistics in the office. When used with a CORS network equipment is set up only once, requiring minimized resources on the field. Provides solutions in the order of centimeters.	Significantly increased equipment cost. When used in base-rover mode, significantly increased logistics for equipment setup. Must have radio connection between base and rover. Must set up base on a known position to use advantages in base – rover mode. When used with a CORS network must have connection to the network, usually via the internet.
Post processing Kinematic (PPK)	Reduced logistics, cost, and complications in the field. Provides fixed solutions in cases where RTK does not. Provides solutions in the order of centimeters.	Corrected data is typically not available until processed. Requires increased resources at the office for processing. Necessary to set up base at benchmark or use a CORS network.
Fast Static	Reduced equipment expense and complication compared to PPK and RTK. Provides more accurate solutions than RTK and PPK Requires less occupation time, typically < 20 min.	Corrected data is typically not available until processed. Requires increased resources at the office for processing. Requires more precise mounting and metadata collection. Necessary to set up base at benchmark or use a CORS network. Provides less accurate solutions than Static.

Survey type	Advantages	Disadvantages
Static	Higher precision than fast static, less equipment than RTK or PPK.	Corrected data is typically not available until processed. Requires increased resources at the office for processing. Requires long occupation times. Requires more precise mounting and metadata collection. Requires increased resources at the field regarding fast static due to the need for concurrent measurements on each roving position. Necessary to set up base at benchmark or use a CORS network
Continuous	Highest possible precision and accuracy.	Requires complex infrastructure, precision mounting, and very long occupation times.
DGNSS	It is useful over a longer baseline (distance between base station and rover receivers) less expensive than RTK.	Less accurate than all other methods
Precise Point Positioning (PPP)	Uses a single GNSS receiver less expensive than RTK provides precise positions with near-RTK errors under good conditions.	Requires more processing power. Requires an outside ephemeris correction stream, which may be available after a long time period.

10.2.1. Static

Static GNSS survey procedures allow various systematic errors to be resolved when high-accuracy positioning is required. Static procedures are used to produce baselines between stationary GNSS receivers by recording data over an extended period of time during which the satellite geometry changes. As such, static surveys rely on long occupation times to produce high-accuracy positions. Static surveys also depend on proximity and availability of CORS stations, or another receiver also mounted over an existing control point (called reference point), receiving observations during the same time period; receivers mounted on these reference points are called bases, while receivers used to measure new points are called rovers. Obviously, a single base can be used to measure numerous points with multiple rovers (Figure 10-2). The precision of the position is a function of the length of survey, the length of the baseline between base and rovers, and also the precision of the mounting and the control point used. Static surveys require much more stable and precise mounts than kinematic surveys. Surveys of first and second order (corridor control) regarding horizontal positions, are conducted with static surveys.

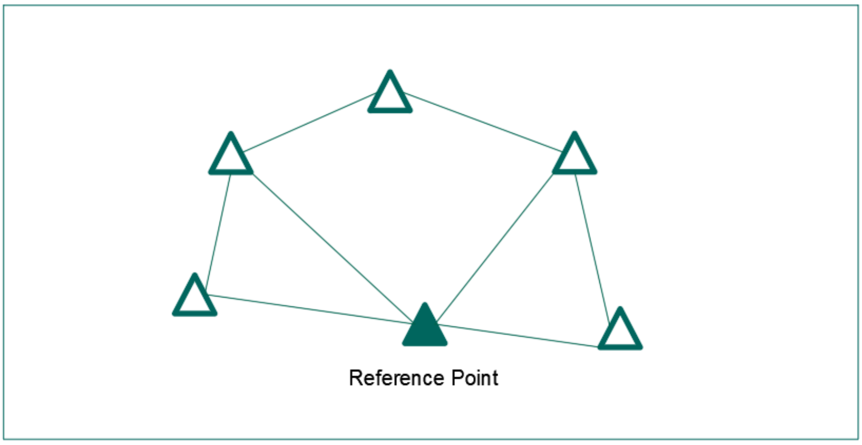


Figure 10-2 Static Survey - Hollow Triangles Represent Rovers

This method is used to:

- Establish, check, and re-establish high order control networks.
- Monitor control networks.
- Monitor movements of the earth's crust.
- Measure deformations and changes in structures and dams.
- Measurement of baselines with large lengths and high accuracy.
- Conventional measurements.

The characteristics and advantages of this method are:

- High measurement accuracy ± (5 mm + 1 ppm).
- Standard method for measuring lengths above 20 km.
- Monitor as many satellites as possible.
- Observation time not less than two hours.
- More accurate.
- Economically better than the traditional method.

10.2.2. Fast Static

In comparison to static GNSS surveys, fast-static GNSS surveys have shorter observation times (approximately 5 to 20 minutes). Fast-static GNSS survey procedures require more advanced equipment and data processing techniques than static GNSS methods. Normally, surveys needing horizontal precision of first order, i.e., corridor control, shouldn't be conducted using fast-static GNSS methods. Fast static surveys also depend on the presence of CORS stations or additional base nearby receivers capable of gathering location corrections concurrently with rover.

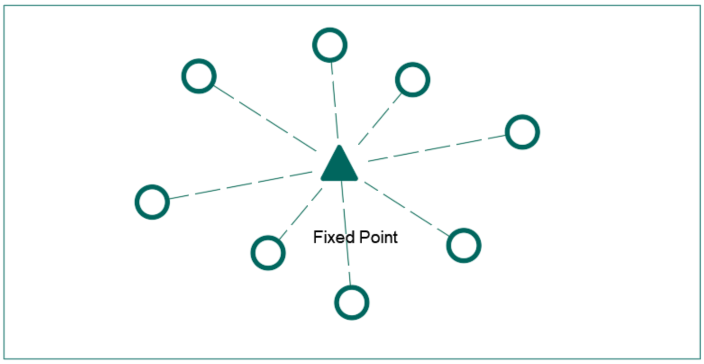


Figure 10-3 Fast Static Survey - Hollow Circles Represent Rovers

On static surveys, a receiver is installed on a point whose coordinates are known (base, on a reference point), while one or more other receivers, i.e., rovers, move to survey new points (Figure 10-3).

Fast static is used for:

- Project control (second order control surveys).
- Control points for aerial survey.
- Detailed surveying works.
- Engineering applications.

Characteristics and Advantages

- Measure of baselines up to 15 km.
- Measurement accuracy ± 10 mm + 1 ppm.
- Monitoring time ranges from 5 minutes to 20 minutes.
- It is an easy, fast and effective method.

10.2.3. Kinematic

A receiver is fixed on a reference point with known coordinates while another receiver (a rover) moves from one point to another. There are several variations of this method, namely, the Stop-and-Go method, its generalization Post Processing Kinematic (PPK), and Real Time Kinematic (RTK). In the Stop-and-Go Kinematic the rover receiver initially remains stationary acquiring observations above the first point for a period of 5 to 10 minutes, and then moves towards the following points and is fixed above every point staying on it for several seconds and so on until the end of the series (Figure 10-4).

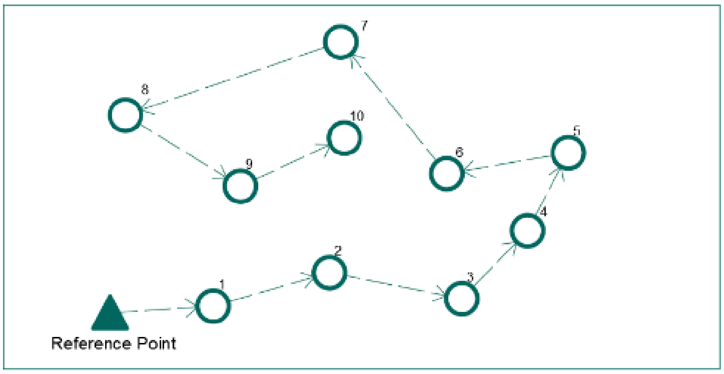


Figure 10-4 Stop-and-Go Kinematic Survey - Hollow Circles Represent Rover Positions

10.2.3.1. Post processing kinematic (PPK)

This method is similar to the stop-and-go method with the difference that the operator does not interfere at the start of surveying because it is done automatically according to the specified time period. Modern receivers support up to 20 kinematic solutions per second.

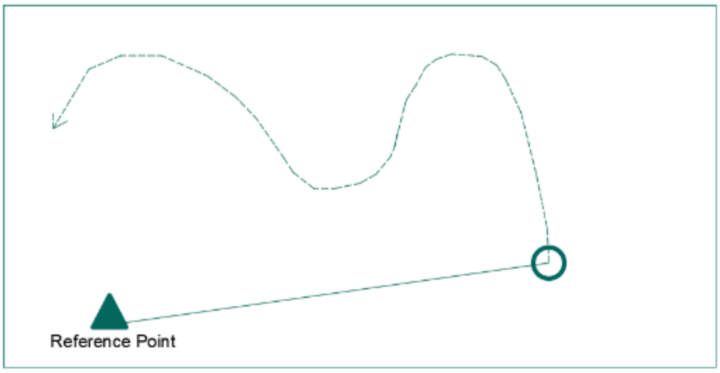


Figure 10-5 Kinematic Survey - Hollow Circle Represent Rover Positions

This method is used in:

- Topographic survey.
- Marine survey work.
- Detailed survey work.
- Construction surveys (roads, pipelines etc.).
- Digital Terrain Modelling surveys.
- Cadastral Surveys.
- Low order control surveys.

The characteristics and advantages of this method are:

- It requires a CORS or a base station on a known control point.
- Rover receiver can be moved by any means, whether on a pole, on the back of the observer or by other transport means.
- Stand at the starting point for a few minutes.

- Automatically measure new points while moving according to a specified time period.
- It is a fast and economical method for continuous measurements.
- Measurement accuracy ± (20 mm + 1 ppm).
- Requires continuous communication with a least four satellites being the same.
- It is a fast and economical method.
- Can be used without the presence of any type of link required by RTK method.
- It may provide fixed solutions in case where RTK fails, since it utilizes data from the subsequent observations (whereas RTK utilizes data only from previous observations).

On the other hand, one may not be ever confident about whether GNSS observations lead to a fixed solution at every survey position.

10.2.3.2. Real time kinematic (RTK)

In a RTK GNSS survey, a base station broadcasts positional correction to a roving GNSS receiver. In order for the user to use the RTK technique, he must establish a data transfer link between a base station receiver and a rover. At least one GNSS receiver over a known point, usually a control survey station, remaining stationary, should be available during the survey. Base receiver can be mounted by the user over an established monument: in this case the range of the corrections is limited due to the means used for transmitting corrections (Ultra High Frequency (UHF) modem), usually less than 10 km.

In the presence however of a CORS, using a base station is not necessary, since CORS stands for this purpose; in this case, range of corrections is limited to 50 - 70 km from the base station depending on the type of the rover receiver. The accuracies obtained are typically in the range ± (10 - 50) mm horizontal and vertical, depending on baseline distance and atmospheric conditions.

This method is considered one of the fastest surveying methods, and follows the same rules and concepts applied in the previously described methods. It is distinguished by the absence of the need of long initialization time periods at the starting point, and the additional need for a viable data link between the base and rover receivers. RTK surveying is used in:

- Topographic survey.
- Marine survey work.
- Detailed survey work.
- Construction surveys (roads, pipelines etc.).
- Digital Terrain Modelling surveys.
- Cadastral Surveys.
- Low order control surveys.

The characteristics and advantages of this method are:

- It requires a CORS or a base station on a known control point.
- Rover receiver can be moved by any means, whether on a pole, on the back of the observer or by some means of transport.
- Automatically measure new points while moving according to a specified time period.
- It is a fast and economical method for continuous measurements.
- Measurement accuracy ± (20 mm + 1 ppm).
- Requires continuous communication with a least four satellites being the same. If communication is lost, a new initialization (typically of a few seconds) is needed.

- It is a fast and economical method.
- Requires a type of link with the base receiver in the form of UHF or Global System for Mobile (GSM) modem. The need for base receiver can be supported by any CORS system, e.g., KSA-CORS.

10.2.4. Differential GNSS

Differential GNSS (DGNSS) is an enhancement to the GNSS which provides improved location accuracy to about 10 - 30 millimeters regarding the best implementations. DGNSS uses a network of fixed ground-based reference stations to broadcast the difference between the positions indicated by the GNSS system and known fixed positions. These stations broadcast the difference between the measured satellite pseudoranges and actual (internally computed) pseudoranges, and receiver stations may correct their pseudoranges by the same amount. The digital correction signal is typically broadcast locally over ground-based transmitters of shorter range.

The configuration of Differential GNSS (DGNSS) and RTK systems are similar in that both methods require a base station receiver setup at a known location, a rover receiver that gets corrections from the base station and a communication link between the two receivers. The difference is that RTK (a carrier phase method) is significantly more accurate than DGNSS (a code-based method).

On the other hand, DGNSS has the advantage of being useful over longer baselines (distance between base station and rover receivers) whereas DGNSS systems are less expensive. The technology required to achieve the higher accuracy of RTK performance makes the cost of an RTK-capable receiver higher than one that is DGNSS-capable only.

10.2.5. Precise Point Positioning (PPP)

The calculation of precise locations and positions can be achieved using also the PPP method. This method can successfully estimate, with errors as small as a few centimeters (ideal conditions), the positions of features of interest. It combines several rather sophisticated GNSS refinement techniques. The main advantage of PPP techniques is that they can be used with near-consumer-grade hardware to yield near-survey-grade results. In contrast to standard RTK methods that utilize a combination of temporarily fixed base and nearby mobile receivers, PPP uses a single GNSS receiver. Because both PPP and DGNSS use permanent reference stations to quantify systemic errors, an overlap between these two different methods is evident.

The major weaknesses of PPP, compared with conventional consumer GNSS methods, is that it takes more processing power, it requires an outside ephemeris correction stream, and it takes some time (up to tens of minutes) to converge to full accuracy. Precise Point Positioning has not yet been adopted at a great extent for surveying purposes, although it seems likely that during the following years it will become ubiquitous methodology.

10.3.GNSS Survey Including the Saudi National CORS Network

The Kingdom of Saudi Arabia Continuously Operating Reference Station (KSA-CORS) network is operated by the General Authority for Survey and Geospatial Information (GASGI). The network currently consists of over 200 active CORS, with new stations constantly expanding the available network (GASGI, 2020).

The aim of the KSA-CORS network is to provide a reliable and accurate Kingdom-wide GNSS positioning service. In addition, the objective of the network is to create, distribute and maintain a national geodetic reference frame called KSA-GRF17.

10.3.1. KSA-CORS Real Time Services

KSA-CORS network provides several positioning services to its subscribed users, on a real-time basis, as described in the next:

- Network PTK: The KSA-CORS network produces the Network RTK correction data stream to users by means of the KSA-CORS NTRIP server. Registered KSA-CORS users can access the correction data stream through the internet, adding the appropriate parameters into their field data collectors / GNSS controllers. The real-time positional accuracy of Network RTK is about 30 - 40 mm.
- DGNSS Positioning: The KSA-CORS network produces the DGNSS correction data stream to users by means of the KSA-CORS NTRIP server. Registered users can access the DGNSS correctional data stream through the internet. The real-time positional accuracy of DGNSS can reach up to 1 m.
- Single Station RTK: The Single station RTK service is available for using in areas where no network RTK service is available. Single station RTK service provides limited accuracy when rover position is more than 15 km away from the CORS station; if it is within this distance, accuracy can be equal to the NRTK service. Single station RTK in higher distances from the CORS base station is possible, at a rather reduced accuracy.

10.3.2. KSA-CORS Products

KSA-CORS network provides several products to its subscribed users, as described in the next:

- GNSS Raw Data Files: The KSA-CORS network provides registered users with GNSS data in several formats as required. Receiver Independent Exchange Format (RINEX) is available in the following formats in versions 2.10, 2.11, 3.02 and 3.03 are supported. These data can be used for phase differential baseline post-processing. Thus, these files facilitate the processing of static, fast static and post processing kinematic surveys. Then, after completing the survey, the user can simply download GNSS raw data either from KSA-CORS web site for their campaign observation time or from a virtual reference station near their site of survey.
- Virtual RINEX: A RINEX file is called Virtual, when raw data have not been really measured by a receiver but have been computed from a real-time network model. The Virtual GNSS files are generated from the network. KSA-CORS virtual RINEX data are derived from surrounding KSA-CORS observations by using interpolation methods.

Virtual RINEX are encouraged to be used when distance from the closest base station is more than 15 km.

- CORS Meta data: Site log files are available for each KSA-CORS station. This file contains information on all the historical equipment (receiver/antenna) used at that station, approximate location, and owner and operating agency, among other information.
- Online GNSS Post Processing.
- Geoid height.

10.4. GNSS Control Surveys

The accuracy of GNSS equipment is not suitable for high precision tasks when working at short ranges. Under ideal situations the accuracy of GNSS equipment can approach 5 mm, where it is common for total station measurements to have a built-in error of only 2 - 3 mm; thus, the total station is a more accurate instrument when working within the total station's maximum range (60 - 500 m). On the other hand, the GNSS equipment can potentially maintain an accuracy of 5 mm, even when performing measurements over the several kilometers. GNSS equipment’s level of accuracy at long ranges is especially useful on large construction projects, such as on highway construction projects covering hundreds of kilometers.

GNSS observations shall be linked to existing control points of the same or higher order of the same control survey; KSA-CORS or any other available CORS, or other control point network used in the Kingdom should not be used for directly establishing new low order control points. Ideally, first order control networks for road projects should be established with static surveys reverencing KSA-CORS regarding horizontal control, and with leveling regarding vertical control; then second order control network should be established with the aim of the respective first order control network and so on, following the specifications of the next paragraphs.

10.4.1. GNSS Control Survey Instructions

This chapter contains standards of accuracy, specifications, and guidelines for the use of GNSS techniques for control surveys. Along with horizontal positions, GNSS also provides heights or height differences with respect to WGS-84 ellipsoid. As such, GNSS leveling requires a geoid model so as to transform ellipsoid height into orthometric ones; throughout the Kingdom the KSA-GEOID21 model has been developed and used for this purpose. While GNSS heighting is a rather economical leveling technique, produced elevations are generally a factor of 2 - 3 times less precise than the horizontal components. Therefore, GNSS levelling can be considered a by-product of the GNSS survey process.

According to Table 3-3 providing the Suitable Methodologies by Control Type and Order, GNSS is mainly used for horizontal control. It is there revealed that control points of third and higher order shall be established by GNSS control surveys regarding horizontal positioning, in conjunction with leveling regarding height control (and trigonometric heighting and static GNSS leveling for third order surveys). GNSS leveling may be useful in preliminary design, but for most detailed and construction purposes only differential levelling and trigonometric heighting shall be adopted.

Among the errors being introduced into GNSS surveys, the most common are multipath errors, referring to the mixture of Line-of-Sight and numerous Non-Line-of-Sight wave components reflected by nearby close obstacles several times before reaching the actual position of the receiving antenna. Therefore, control points established and measured with GNSS techniques should be located in a clear sky area, away from natural or man-made features that may cause this type of error.

Table 3-3 establishes the criteria and specifications for GNSS control surveys, in addition to the following information about the field and office procedures to be followed:

1. All receivers and baseline processing software shall be of the geodetic type.
2. Requirements regarding instrument calibration posed in Section 4.2 for all topographic equipment, are also applied in GNSS surveys. If necessary, hardware and software verification can be performed using high order available control points. Another useful method is the "zero base line" measurement, which can be accomplished by connecting one antenna to two receivers using an antenna splitter. The location data obtained from both receivers must correspond within a fraction of 10 mm.
3. All equipment used must be well controlled, maintained and operated by specialized and trained personnel.
4. All GNSS measurements depend on the exact position of the antenna, therefore, centering it above the ground mark accurately gives the best results.
5. The locations and geometry of the satellites during field surveying shall be sufficient to ensure accurate results. The effect of the satellite distribution, on the Geometric Dilution of Precision (GDOP) should not be more than 6, and, on the Position Dilution of Precession (PDOP) should not be more than 8. The user should comply with the GNSS receiver's manufacturer recommendations for the accuracies achieved during the surveying phases. This helps to solve unknown integer ambiguities problem when using the respective GNSS processing software.
6. Starting work with inaccurate coordinates adversely affects the accuracy of the baseline results. Initial coordinates should be used within 10 meters of the real receiver position to start processing and adjusting the baselines. After stopping the selective availability and with the improvement of geodetic receivers, this requirement regarding initial locations is achieved in the vast majority of surveys.
7. It is not necessary to obtain meteorological readings, since modern GNSS receivers and baseline processing software are used to model the lower atmosphere.
8. When re-centering the receiver, the antenna height must be changed by at least 0.1 m, unless it is installed on a concrete base.
9. Multipath may be a troublesome source of errors, especially when measurement periods are short. Typical conditions for multipath error encounter are: proximity to iron roofs, damp trees, tall buildings, and wire fences. If there is a possibility of multipath error, the measurement period shall be increased by a factor of 2 to allow reducing its impact with changing the locations of the satellites.

Regarding GNSS leveling used in lower order surveys according to Table 3-3, the following should be taken into account.

1. KSA-GEOID21 shall be used for converting WGS 84 ellipsoid heights to geoid heights and vice-versa.
2. GNSS observations shall be linked to existing control points of the same or higher order of the same control survey; KSA-CORS or any other available CORS, or other control

point network used in the Kingdom should not be used for directly establishing new control points with GNSS leveling.

Table 10-2 Suitable Standards and Specifications for GNSS Control Surveys (MoMRAH, 2005) (ODOT, 2014)

Description	**Order of Survey**				
	1st	**2nd**	**3rd**	**4th**	**5th**
Standard Deviation (mm)	1	3	8	15	30
GNSS Surveying Methodologies					
Static	✓	✓	✓	✓	✓
Fast Static		✓	✓	✓	✓
Kinematic				✓	✓
Surveying Process					
Minimum elevation mask (degrees)	15	15	10	10	10
Minimum number of satellites to be observed	5	5	5	4	4
Maximum centering error (mm)	3	5	5	5	7
Minimum number of independent measurements of the antenna height	2	2	2	2	2
Maximum GDOP during observations	8	8	8	8	8
Maximum PDOP during observations	6	6	6	6	6
Network Geometry					
Recommended minimum distance between points (km)	5	0.2	0.2	0.1	-
Typical distance between points (km)	10	0.5 - 1.5	0.2 - 0.5	0.05 - 0.20	-
Max distance between points for single frequency (km)	-	-	3	2	1
Minimum number of points to be in the network interior	-	4	3	3	2
Independent Point Centering					
At least 3 times (%) of the total points	5 %	4 %	2 %	1 %	-

Description	Order of Survey				
	1st	2nd	3rd	4th	5th
At least twice (%) of the total points	1 %	1 %	8 %	5 %	-
Least independent base line at each point	3	3	2	2	2
Static Surveying Requirements					
Minimum number of simultaneously working receivers	6	4	3	2	2
Recommended time interval between readings (sec)	5	5	5	5	5
Minimum number of baselines for each point	3	2	2	2	1
Minimum number of simultaneously visible satellites	6	6	5	5	4
Minimum Occupation Time (hours)					
Single frequency, Baselines < 5 km	-	-	-	1.0	0.5
Dual frequency, Baselines < 5 km	2	1.5	1.0	٠٫٥	٠٫٢٥
Dual frequency, Baselines 5 to 10 km	4	3	1.0	-	-
Dual frequency, Baselines 10 to 15 km	6	6	-	-	-
Dual frequency, Baselines 15 to 30 km	8	-	-	-	-
Dual frequency, Baselines > 30 km	12	-	-	-	-
Ephemeris					
Minimum ephemeris requirements: BE = broadcasted RE = observed half of the International GNSS Service (IGS) Ultra-Rapid ephemeris	RE	RE/BE	BE	BE	BE
Repeated Baselines					
Max average difference (mm) for any baseline $(\Delta X, \Delta Y, \Delta Z)$	0.1	1	5	10	50
Maximum difference (mm) for any baseline $(\Delta X, \Delta Y, \Delta Z)$	-	1.5	8	15	75

Reference control points (first to third order) should be surveyed using static or fast static surveying methods. When using kinematic, e.g., RTK, methods for creating control points (fourth and lower survey orders) the following procedures should be followed:

1. A project control system should be created or verified.
2. Base stations shall be installed over control points of third and higher order.
3. The measurement of the new control points should be checked at least two or three times.
4. A restart in the rover receiver should be made between measurements with the use of different antenna heights. These restarts help to find the correct solutions to the ambiguities and make some errors random.
5. Rover receiver should be placed at least on pole with the aim of an additional tripod, or on a tripod-tribrach configuration. Accurate receiver centering, horizontal adjustment and measurement of the antenna is very important.
6. Tilt measurements even with appropriate IMU enabled receivers are not allowed for control surveys.
7. Care should be given to correctly measure the antenna height with respect to receiver's Antenna Reference Point (ARP), according to the manufacturer's specifications.
8. Surveying a new control point should be performed in no less than 30 seconds. The positioning accuracy should be less than 0.02 m, the cut-off angle should be no less than 15 degrees, and the number of satellites should be no less than five.
9. Point accuracy set at 0.02 m results at an accuracy of 0.05 (x 2.45) with 95 % CI for easting and northing.
10. The compatibility with the existing control points (horizontally) must be less than 0.05.

10.4.2. GNSS Control Surveys Planning

One of the first steps in GNSS survey planning is selecting the appropriate technology for the required degree of accuracy. Table 3-3 provides basic information while Table 10-2 shows instructions for users about the techniques that can be used to establish control surveys according to survey orders. The location and distribution of the established control points do not depend on factors like network shape or control point mutual visibility.

High order surveys are in general conducted with static and fast static methods producing baselines. With 3 receivers observing simultaneously there are 3 possible baselines; and 4 receivers will produce 6 possible baselines. Formally, for a given group of N receivers collecting data simultaneously, there are M potential baselines:

$$\boldsymbol{M} = \frac{\mathbf{1}}{\mathbf{2}} \times (\boldsymbol{N} \times (\boldsymbol{N} - \mathbf{1})) \qquad \text{Equation 10-2}$$

However, several of the *M* potential baselines are dependent because they have been derived from the same observational data as the independent baselines. Alternatively, given that the independent baselines have been computed by the processing software, there are no more unknowns left; processing the remaining dependent baselines only adds non existing redundancy resulting to falsely optimized adjustment statistics. The number of independent baselines available for any single GNSS session produced by N receivers is N-1. Therefore, in

Figure 10-6, 4 receivers produce 3 independent baselines. During post-processing presented in Section 12.3 of this Volume, the surveyor can elect the independent baselines for that observation session.

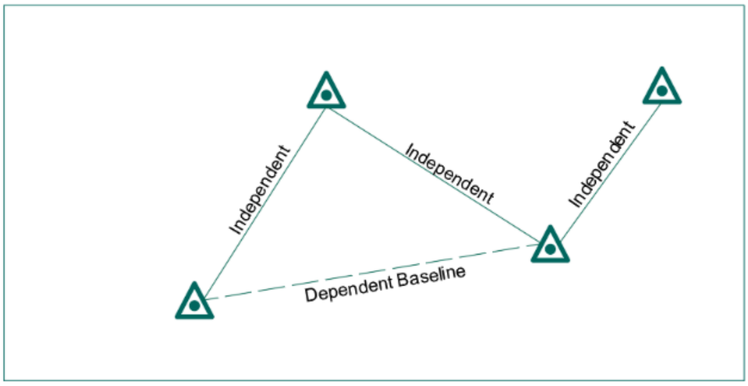


Figure 10-6 Independent and Dependent Baselines

The network geometry determines how errors are distributed, and the quality of the final control point coordinates. Therefore, the selection of observation baselines is of high importance. Dependent baselines are not to be included in the network adjustment. At least two network points should be connected to each point, which needs to be occupied at least twice using an independent setup. This requires rearranging receiver's setup on the tripod/tribrach, and re-measuring the instrument height. Figure 10-7 reveals that the network should consist of a series of interconnected, closed loops, avoiding having hinged stations that could serve as joints. All adjacent stations should be connected using the "20 % rule" (ODOT, 2014), stating that an independent baseline should be measured connecting points that are closer than 20 % of the total distance between those points traced along existing baselines (Figure 10-7). At a minimum, three geodetic reference stations belonging to KSA-CORS must be used to control the first order network.

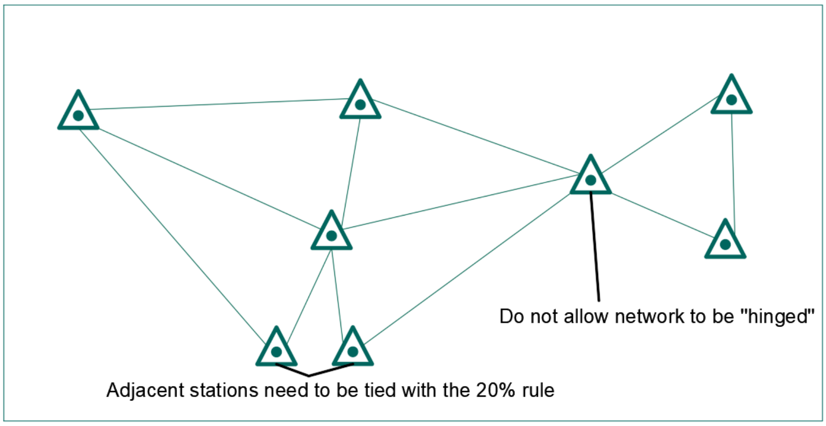


Figure 10-7 Control Network Best Practices

In addition to the information in Table 10-2, the following instructions shall be followed:

1. All first order survey works shall be linked to KSA-GRF17 and KSA-VRF14, using KSA-CORS.
2. At a minimum, three geodetic reference stations belonging to KSA-CORS must be used to control the network.
3. At least two network points should be connected to each point with independent baselines; each point needs to be occupied at least twice using independent setup.
4. Just like TPS surveying, accurate antenna centering and horizontal adjustment using the leveling screws of the tribrach is obligatory. Accurate measurement of the antenna height is also very important. After enough time has passed, an independent centering and horizontal adjustment of the antenna and measurement of the antenna height shall be performed.

After acquiring information about existing control networks in the area, of the same or higher order, a design of the control point network to be established should be performed. Given the variety of the equipment used and the fact that during filed surveys the monitoring window time is short, the lack of attention to good planning may turn some simple obstacles into unpleasant surprises during the monitoring work.

The planning steps for survey work can be summarized as follows:

1. Network Design: It includes defining the scope of the project and the network, which depends mainly on the requirements of accuracy and the shape of the satellites to be monitored, in addition to the network of existing control points. It follows that the design of the best combined formation between the new and existing points gives the best accuracy regarding relative positions.
2. Schedule Surveying Times. When determining the surveying schedule, the following should be taken into account:
    a. KSA-CORS or existing control points will be used.
    b. The number of available receivers.
    c. Planned and traveling time for each point.
    d. The number of points per day.
    e. The occupation times.
3. Monitor Quality. The following should be taken into account when selecting the receivers:
    a. Choosing the type of receiver will affect the type of recording system and media.
4. Exploration: Planning cannot be considered complete until after the field exploration process. The following must be done during the exploration process:
    a. Determining land type and indicating the possibility of vehicles running on it.
    b. Determine required cable lengths.
    c. Identify natural and man-made obstacles in the region.
    d. Determine locations of plants and trees that may hinder or cause errors during surveying.
    e. Review the condition of existing control points. Ensure their presence making sure they are not destroyed.
5. Organizing Administrative Affairs: From a practical point of view, the desire is always to reduce the cost of field work without compromising the required accuracy. The reduction is focused on the following elements:

   a. Transportation and movements.
   b. Numbers and types of available receivers.
   c. Use of owned or rented receivers.
   d. Use of available CORS networks.
6. Other factors such as:
   a. Determine the type of electrical source, is it alternating current, or 12-volt DC or 24 volts.
   b. The quality of the devices, their durability, weight and portability
   c. Available cable lengths and external power sources.
   d. The nature of the earth's surface, its walkability, and the accessibility of existing control points.
   e. The appropriate means of transportation for the project area and necessary for transporting equipment and personnel (cars / transport vehicles / helicopters).

11. GNSS Data Collection

11.1.Data Collection Principles

The abundance of data may be an indication of the accuracy with which the measurement was made. For this indication to be meaningful, the inclusion of potential error sources should not be repeated in a systematic manner in the available measurements. In GNSS surveying, there must be multiple surveying works with alternative satellite geometries and frequent point surveying, so as to eliminate potential sources of cumulative errors resulting from multipath, orbit deviation, and delay in the upper and lower atmosphere. Even if the point measurements are repeated on another day, the data must be collected at multiple points in time so that a different configuration of the satellites can be obtained. Abundant observations also provide additional validation for positioning errors along with a second set of antenna height measurements.

Any equipment used during GNSS surveys to center the antenna above the control point (e.g., tribrach, pole, horizontal bubble etc.) shall be tested periodically and adjusted if necessary. These tests may be carried out by repair and maintenance professionals or by the surveyor himself. In either case, the test results must not exceed the maximum permissible errors, and the test documents must be attached to the surveyor's report.

The observational errors (weights) used in least squares adjustment shall include estimates of the total antenna centering errors, which consist of:

- Centering accuracy in the aforementioned test.
- Error in antenna height measurement.
- The stability of the center of the antenna base.

Regarding GNSS control surveys, there should be an independent centering and data collection method for each baseline measurement, and antenna heights should be measured at the beginning and the ending of each independent setup. If the antenna height measurements are taken from the ground level instead of the level of the specified mark, then at the beginning or at the ending, the monument’s height must be recorded for at least three points around its ground level. It is recommended to use fixed height centering equipment for all surveying work, especially for vertical adjustment projects. Clear pictures and detailed sketches of the monument/point mark should be taken at setup. Notes must be prepared for each point, including at least the following:

1. The start and end times of the occupation.
2. The name of the surveyor based in the station.
3. Identification of the receiver (model and serial number).
4. All antenna height measurements (in meters).
5. Point definition (point name and/or number).
6. A description of the monument and mark.
7. A sketch and a photo of the monument / new control point.

The duration of the GNSS surveying varies due to several factors, including the following:

- The level of accuracy required.
- Number of available satellites.
- Available receivers.

- Surveying technique.
- Processing software.

It suffices to say that sufficient results can be obtained for surveying periods ranging from one minute to several hours or days. The abundance of observations with appropriate analysis and documentation will reduce any problems. The actual errors are reflected in the network adjustment when an appropriate weighting strategy is used with abundant observations. Table 10-2 shows additional specifications for planning and data collection.

The antenna height (Figure 11-1) between the point mark and the antenna base center is measured before and after each monitoring cycle and recorded to the nearest 1 mm, and the antenna must not be moved during the occupation. To prevent any interception of the satellite signals, the antenna is placed in unobstructed locations higher than 10 to 15 ° from the horizontal horizon.

Accurate GNSS observations depend on clean and unaltered signals. Nearby objects like buildings, trees and vehicles can reflect the GNSS signals resulting into multipath effects and cause measurement errors. Strong radio transmissions from handheld radios, radio towers, GSM antennas or power lines may also degrade the GNSS signals. Avoid such areas when possible. An open view of the sky allows more satellites to be tracked by the receiver.

Back-to-back observations on the same point should be considered “independent.” In order to achieve the proper second occupation, the setup should be off-leveled, the tribrach turned 120 °, and re-leveled with a new antenna height. Taking these steps will help to reduce systematic error and blunders. The time it takes to off-level and re-level is enough time for a gap between observations.

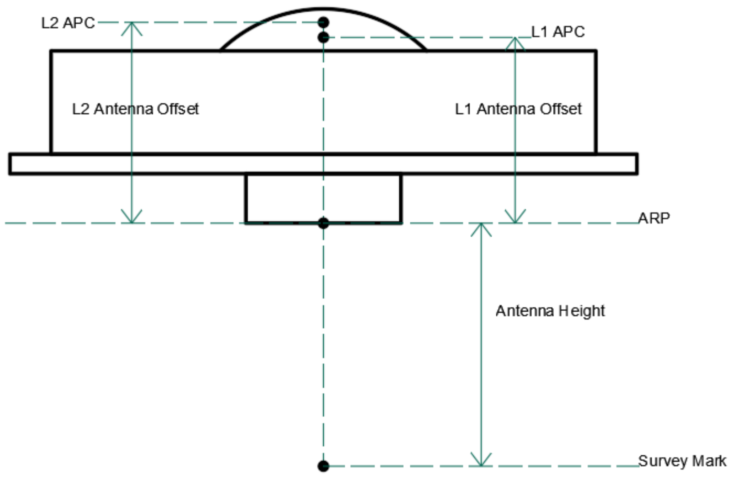


Figure 11-1 Antenna Reference Point (ARP) and Antenna Phase Center (APC) for L1 and L2 and Offsets

Blunders in antenna height measurements are the most common source of error in GNSS surveys. It is critical to detect these blunders because all GNSS surveys are three dimensional whether the vertical component will be used or not. Antenna height measurements determine the height from the survey monument mark to the ARP of the GPS antenna (Figure 11-1).

Modern receivers have two APCs, one regarding L1 signal and one regarding L2 (and two offsets, respectively). With the exception of fixed-height tripods and permanently mounted antennas, independent antenna heights should be measured at the beginning and end of the observation session.

The following instructions deal with different types of GNSS surveying, such as Fast Static, Static, RTK, and PPK Stop/Go. Users shall additionally follow the procedures and recommendations set out in the equipment and software manual.

11.2. Baselines for Static Surveying

The following guidelines are applied when surveying with normal static method:

1. The recording time should be at a minimum of 2 hours, increasing with the length of the surveyed baseline.
2. The locations of the satellites should change significantly during the monitoring period.
3. As many satellites as possible should be surveyed, with a minimum of four being common to all rovers used in the static survey, at the same time period.
4. Dual frequency (L1/L2) receivers should be used. Single-frequency (L1) receivers can be used for short baselines (less than 10 km) and for lower order surveys.
5. Sufficient observations should be collected to resolve ambiguities, especially for baselines more than 15 km.
6. Just like TPS surveying, accurate antenna centering and horizontal adjustment using the leveling screws of the tribrach is obligatory. Accurate measurement of the antenna height at the closest 1 mm is also very important. Care should be given to correctly measure the antenna height with respect to the ARP (Figure 11-1), according to the manufacturer’s specifications.

11.3. Baselines for Fast Static Surveying

The following guidelines apply when using fast static method:

1. Sufficient data must be collected to resolve ambiguities.
2. According to the receiver used, manufacturer's recommendations should be consulted regarding length of observation periods, numbers and locations of satellites, and suitability of single - or dual-frequency receivers.
3. Multipath errors, caused by reflection of nearby close obstacles, may be a major source of errors, especially during short monitoring periods, so attention should be given to this issue.
4. The recording time should be at a minimum of 15 minutes, increasing with the length of the surveyed baseline.
5. Accurate antenna centering, horizontal adjustment using the tribrach’s leveling screws and measurement of the antenna height with respect to its ARP is very important.

11.4. Baselines for Kinematic Surveying

These guidelines are applied to the baselines for stop-and-go and PPK surveys:

1. When navigating between new points, rover receiver has to be continuously on, and not lose communication with the satellites.
2. It is preferable to have five or more common satellites due to the possibility of losing signals during the movement between points.
3. The minimum time to stay on a point should be enough to record 5 to 10 solutions, depending on the receiver.
4. Multipath phenomena may be a major source of errors, especially during short surveying periods, and attention must be paid to this issue.
5. Single frequency geodetic receivers can be used for base-rover distance less than 10 km, although dual frequency receivers are highly preferred

RTK surveying technology includes transmitting radio data from a basic fixed receiver to mobile receivers and processing this data in real time to obtain three-dimensional coordinates for the mobile devices. The following guidelines apply to RTK surveying:

1. Single-frequency geodetic receivers, can be used for distances up to 5 km from the base; however, dual-frequency receivers are highly preferred.
2. The typical range for RTK survey can be up to 50 - 70 km depending on the type of rover receiver.
3. Surveyor should comply with the manufacturer's recommendations regarding the minimum number of satellites and their location to minimize the effect of satellite distribution on the positioning accuracy.
4. Multipath effect may be a major source of errors especially in kinematic methods. Care shall be taken in this matter, providing that base and rover receivers are placed in areas where multipath phenomena are reduced. If multipaths are likely to be present at the rover, the surveying period for each point should be increased so that the multipath effect is minimized during changes in satellite positions.
5. In order to allow sufficient variability in the pool of satellites being used and to improve error detection such as multipath, base repositioning should be done only after 45 minutes, or more, have been passed on the reference point.
6. When moving the base station to a new reference point, several observed points from the previous base should be surveyed again (twice) independently; this is the most reliable way to check the absence of cumulative or major errors. Section 12.3.4 demonstrates this method used for analyzing the RTK survey quality, employing closure error comparisons between multiple surveys of the same point.
7. Least of squares adjustment cannot be applied to RTK surveyed points. However, some RTK receivers may allow users to take down baseline components (instead of coordinates) for inclusion in a least-square adjustment.
8. Base receiver positions and coordinates should be known in advance before starting the RTK survey (or use a CORS).
9. Additional existing control points should be surveyed during GNSS RTK surveys to ensure adequate compatibility with the horizontal and vertical reference points in the project area.
10. When IMU tilt measurements are available, depending on the receiver, manufacturer’s instructions about IMU calibration should be followed before taking a tilt measurement. These instructions include taking the pole and draw a circle towards the ground, shaking the pole and walk around, waiting etc.

11. When IMU tilt measurements are available, depending on the receiver, manufacturer's specifications about allowed tilt angles shall be followed.
12. The following items must be recorded with the extracted coordinates: definition of reference points, date, time and reference, number of observed satellites, and standard deviation of the extracted coordinates, pole height, code.

One of the most important advantages of the RTK method is that there is no need for further data processing, and therefore the required quality is reached without applying any additional calculation step.

11.5. Field Notes and Data

General documentation requirements are given for all survey works in Section 4.4 of this Volume. In addition, the following points should be taken into account with regard to field notes and collected data for GNSS survey works:

1. Field notes shall be used to record all relevant information, and may be handwritten or electronic. When using handwritten notes, a field book shall be employed.
2. The type of receiver, its serial number and the program used for data collection shall be recorded in the field notes.
3. It is necessary to have evidence of a complete independent review of the antenna height.
4. The field notes must include narrations of the surveyed part of the network in addition to the name or identification used for each point, and any baseline being measured must be fully explained.
5. Field notes in the form of electronic means or field registration book must be available to any competent authority for examination.
6. Preliminary survey data must be saved in the archived records to be available upon request for review by the supervising authorities.
7. If the supervisors request the result files of the baseline surveys and final adjustments, they must be submitted in digital form.

12. GNSS Data Processing

12.1. Introduction

GNSS data processing includes the collected data, CORS data obtained by e.g., KSA-CORS, other data recorded in field notes, e.g., antenna heights, processing stage measurements for identifying baseline vectors and performing adjustments and transformations for the processed vectors and locations. Each of these steps requires analyzing and controlling the quality of the surveyed data using statistical measurements and assessments to obtain the required level of reliability. These steps also depend on the measurement technology, the type of GNSS receivers and antennas, the recorded observations and the processing programs used. Factors affecting data processing include:

1. Initial Position Accuracy: The absolute coordinates of the location of the reference point forming the baseline must be referred to KSA-GRF17 and KSA- VRF14, and their horizontal and vertical coordinates must be known and their accuracy be as shown in Table 10-2. As a rough estimate, if there is an error of 10 meters in the initial unknown point coordinates, an error of approximately 1 ppm results in the calculated baseline.
2. Orbit Ephemeris: Broadcasting orbits may be used to process all baselines of less than 30 km in length, given its suitability for the selected survey order (Table 10-2 regarding Ephemeris). Given that an orbital error of 20 meters results in a baseline error of approximately 1 ppm, International GNSS Service (IGS) Ultra-Rapid (observed half), or Rapid ephemeris should be used for such baselines.
3. Atmosphere Error Reduction: Standard models for stratospheric GNSS signal delay can be used, as employed in the majority of commercial baseline processing software employ.

Details about baseline processing are given in the next sections.

12.2. Baseline Processing

Baselines are processed with appropriate software, depending on the receiver's manufacturer, data provided, and surveyor's preference. Given the development of RINEX files, the vast majority of geodetic receivers produce such file exports, enabling them to be processed by any vendor's software. Available CORS like KSA-CORS also provide RINEX files in several formats. The post-processing software employed should be able to perform constrained least-squares adjustment producing statistics for baselines and network errors: calculate baseline residuals, produce error ellipses, compute degrees of freedom, etc., so as to provide surveyor the ability to determine the quality of the final coordinates and the source of any potential error.

Upon importing observation data, is important to ensure that the antenna type is correct and antenna height is also included in the file. Several software interprets the RINEX file header applying a vertical offset on the Antenna Phase Center (APC), i.e., the actual antenna position inside the receiver (Figure 11-1). Surveyor should be aware of this offset and correct it appropriately in case software misinterprets offsets and antenna height. Absolute receiver antenna models from IGS have to be used together with correct phase center offsets in order to exploit full consistency.

A fixed integer solution is obligatory for every baseline processed and included in the network. Commercial GNSS processing software determine the integer ambiguity, i.e., the unknown number of full wavelength cycles that exist between the antenna and the satellite, of the GNSS satellite signals. When the integer ambiguity turns to “fixed”, this means that the software has calculated the exact cycle number. In any other case, the final coordinate solution turns to “float”. Noisy signals or short occupation times will prevent the ambiguities from being fixed.

Baselines with problems may happen even with properly occupied points, with long occupation periods, multipath free environment etc. In this case, surveyor should look for cycle slips in the baseline data; cycle slips occur with a loss of lock on a particular satellite and show up as breaks in the particular satellite’s graph. Cycle slips happen due to obstructions, low signal strength, higher nearby radio signals, etc. If several breaks occur in the graph, the satellite should be eliminated and the baseline has to be re-processed. Lower quality satellite signals should be considered for removal for baseline processing, by raising the cut-off angle and reprocessing them. Float solutions are excluded from the final solution using least squares solutions (ODOT, 2014).

Accurate coordinates can be obtained only when orbit information for the used satellites contained in their ephemerides is known; each individual satellite broadcasts its predicted ephemeris, containing orbit parameters that predict where the satellites will be for the next several hours. Though broadcasted ephemerides data have been greatly improved in recent years, in several situations especially in long baselines, a more accurate “observed” ephemeris can be used. There are several types of ephemerides from the real time broadcasted to the final being available after 12 - 18 days. Among them the observed half of the Ultra-Rapid ephemeris is sufficient enough for most project control work, available after 3 to 9 hours (ODOT, 2014). The following guidelines shall be employed when processing baselines (MoMRAH, 2005), (ODOT, 2014):

1. The horizontal and vertical coordinates of the reference point shall be provided in the KSA-GRF17 and KSA- VRF14. Reference points shall be of equal or higher degree.
2. Due to the influence of the stratosphere, dual frequency receivers are used for long baselines. When a single frequency is used, baselines should not exceed 5 km.
3. All processed baselines shall be ambiguous fixed solutions, irrespective of their length.
4. The processed baselines must be inspected for poor quality. If data quality is insufficient the baseline must be excluded from the processing and observations have to be repeated.
5. Satellites with cycle-slips should be removed before baseline processing.
6. The minimum elevation mask (cut-off) angle is 10 ° and 15 ° depending on the survey order.
7. Best practice requires that baselines are processed with the IGS Final, Rapid or Ultra Rapid ephemerides. However, if this is not practical, baselines less than 10 km for second and lower order control surveys may be processed with broadcasted ephemeris.
8. IGS tested Absolute Antenna Models should be used where possible.
9. The software default atmospheric models can be used for processing of all baselines.
10. The name of the processing software, the version number and baseline processing options used during processing should be documented.

12.3. Error Adjustment

12.3.1. Horizontal Adjustment

Horizontal adjustment shall be made with the least squares method to analyze the baseline data and determine the surveyed points' coordinates. The software used for the adjustment should be able to calculate the standard errors from different baseline statistics, and include models for KSA-GRF17, KSA-VRF14, and KSA-GEOID21. The software must be also able to contain the reference points' constraints as weighted observations.

When using a weighting strategy for least squared adjustment, the same weighting values shall be used for all similar baseline solutions, regardless of using minimally of fully constrained adjustment. Baseline weights are obviously controlled by their length, time of occupation and other statistical values determined by the baseline processing software.

Adjustment results should be supported by statistical analysis to detect any significant errors. Accuracy of coordinate calculations is estimated by error propagation methods from the control points. The coefficient of variation is used to control estimates of resulted point accuracy. It has been shown that an increase in the coefficient of variation above 1.5 specifically indicates the presence of cumulative errors between the survey and the network. In this case, the respective chi-square test fails and the new survey is rejected.

The baseline processing software must be able to process the network with least squares adjustment, or, at least, to provide statistics for the baseline vectors so that they can be entered into another network adjustment software. The steps for producing the final network adjustment include a minimally constrained and a fully constrained adjustment as described in the following sections.

12.3.1.1. Minimally constrained adjustment

A minimally constrained (or inner-constrained) adjustment is a least squares adjustment using only one point of control, with a single baseline connecting it to the unknown points, thus allowing thus the measured network to reveal its internal reliability and accuracy. An initial minimally constrained least squares adjustment must be made to extract control points coordinates, according to the following:

1. A chi-square test that determines whether there is a statistically significant difference between the expected deviations and the actual observations shall be performed for accepting adjustment results.
2. After completing the least squares adjustment, the expected positional error along with the respective error ellipses of the final surveyed points, and baseline residuals shall by computation of positional errors shall consider a unit weight (usually equal to 1), unless the estimation of the chi-square test fails; in the latter case, the observations, the statistical or even the mathematical model have to be checked, following be a reset in the parameters. In the case of an inability to treat the situation, the estimated error in the baselines have to be increased by the coefficient of variation.
3. To confirm the quality of the observations, the baselines residuals should be reviewed to ensure that there are baselines with residuals that have to be handled or removed. A review of statistics often includes a careful assessment of the standard deviations in baselines. A check for possible outliers among the baseline residuals has to be

performed. Any outliers should be removed, or apply a different weighting strategy, and adjustment has to re-run until all observations are within the defined tolerances of accuracy and reliability.

4. To comply with the requirements of internal reliability, the following conditions must be fulfilled:
   a. The error ellipses must confirm the ability of the network design to meet the specifications.
   b. The standard residuals, together with the variance estimates, should confirm that the observations fully agree with the required order of survey.
5. All survey control points must conform to the specifications of the appropriate survey order, whether or not these points are linked to baseline observations.

12.3.1.2. Fully constrained adjustment

After the minimally constrained adjustment has been completed, a fully constrained least-squares adjustment shall run, including positions of all known stations, along with all the respective independent baselines. This adjustment is subject to the same steps as above. Again, the chi-square test is performed, the positioning errors and error ellipses are calculated and the solution is accepted given that the chi-square test has passed and all point errors are within the accepted range. Possible outliers shall be checked among the control points and their baselines. Outliers have to be removed and the constrained adjustment has to re-run until all control points are within the defined tolerances of accuracy and reliability. Final baseline residuals have to be checked of being within the accepted range determined by the survey order. In the other case, the survey is rejected and the total survey shall be repeated.

12.3.2. Vertical Adjustment and Control

Elevations resulting from survey are determined on the basis of the compatibility of this survey with existing elevations. Given that GNSS provides elevation information with respect to WGS84 (ellipsoidal heights), the presence of a geoid model to convert the ellipsoidal heights to orthometric, i.e., geoid, heights, is required. According to (MoMRAH, 2005) there are three geoid modeling strategies, using the least squares adjustment methods to determine the orthometric height, using control networks that have been previously appropriately established. The reference network should be free from large and cumulative errors, and only local vertical adjustment and geoid differences remain to be resolved. The referenced in (MoMRAH, 2005) geoid modeling strategies are single height, two-height, and least squares adjustment.

Fortunately, with the development of KSA-VRF14 and KSA-GEOID21 a valid geoid model covering the total Kingdom has been established. Moreover, given that first and second order vertical control in survey works for road projects as in this code, is provided by leveling, the significance of vertical adjustment with GNSS degrades.

As such, vertical adjustment with GNSS static surveys is performed simultaneously with horizontal adjustment, with the corresponding baseline processing and network adjustment software, and calculated WGS84 ellipsoid heights are simply turned into geoid heights employing the KSA-GEOID21 model (and vice-versa).

12.3.3. Least Squares Adjustment Analysis

Careful analysis of the least squares adjustment output is the basis for the completion of a project that achieves its desired objectives. The following statistics must be evaluated for each adjustment made in any project:

1. The variance of the unit weight in the network (the coefficient of variation) and the degrees of freedom must be evaluated. Values of coefficient of variation from 1.5 to 1.0 are used as an acceptance statistic for the control survey.
2. The highest and lowest values of the root mean square (RMS) and standard deviation of the absolute observation residuals must be equal or better than the required accuracy. The algebraic sign of the residuals at a point or loop in the network may indicate that there are cumulative or large undetected errors in the observations that distort that part of the network, and that any common trends in the residuals are a reason for further testing.
3. Standard residuals, calculated from absolute residuals divided by their standard error, should be compared with the chi-square test, and any standard residuals in excess should be investigated. Failing to pass the chi-square test is an indication that some error has occurred in the model. These tests are greatly influenced by the network's external constraints. It is unusual for networks of high quality to fail the chi-square test and isolated exceeding residuals are not considered as a justification for rejecting the control.
4. Errors ellipses at a 95 % CL should be calculated for point coordinates and for each baseline. Error ellipses give a useful estimate of points' reliability. Errors ellipses radii are multiplied by the coefficient of variation in the case it is higher than 1.5. Semi-circular and small ellipses are a sign that the network is implemented according to the conditions, while ellipses of large and irregular errors indicate the presence of problems in the baselines, or weaknesses in the network design.
5. The apparent displacements of the site must be calculated from the differences between the coordinates obtained from the minimally constrained adjustment and the known values of the external control points of the network. This is used to check cumulative errors before calculating the final adjustment and assign temporary accuracy settings to the project.

12.3.4. Closure Error Analysis for RTK

In RTM GNSS surveying techniques, the coordinates of the new points are calculated by radiation from a single reference point; the same also applies to fast static when using only two receivers. With such a technique, there may not be enough direct measurements between the new points, and least squares adjustment may not be suitable to be applied. In this case, a closure error analysis applied with the aim of two independent bases can be applied (MoMRAH, 2005).

The best way to explain the advantage of using closure error analysis, instead of least squares adjustment, can be illustrated by the example shown in Figure 12-1, which is a survey project where the customer requests the surveying of new control points (from A to E) along the axis of the road. The client also requests a relative horizontal accuracy of 0.050 meters (at 95 % confidence) between the new points, and that the coordinates of all points be of fourth order.

All new points were initially surveyed using point S1 as a reference point, and then surveyed again using point S2 as a second reference point.

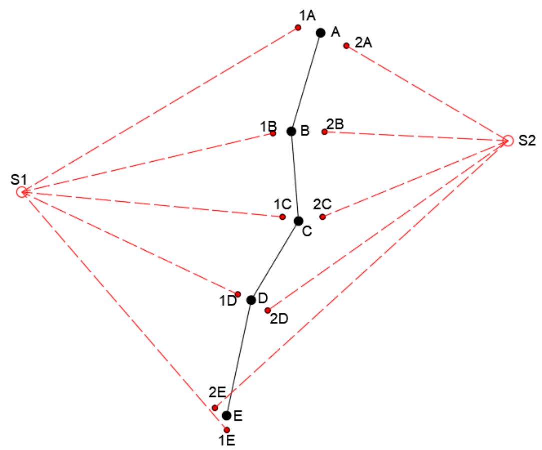


Figure 12-1 RTK Closure Error Analysis

For example, when surveying the new point A from the reference point S1, coordinates 1A for point A are produced; then, when using S2 as reference point, the coordinates obtained for point A are 2A. For the test, it is necessary to take into account the closure error between the surveyed positions on each new point (e.g., between 1A and 2A). The standard deviations SD_{A1} and SD_{A2} of this error for each observation A1 and A2 can be calculated according to the observed GNSS baseline expected error, which is given by the following equation:

$$\boldsymbol{SD_{baseline}} = (\boldsymbol{C_1} + \boldsymbol{C_2} \times \boldsymbol{d}) \qquad \text{Equation 12-1}$$

where variable C_1 (constant) summarizes errors regarding inaccuracies of the receivers etc., while C_2 (scale factor) includes errors proportional to the baseline length between the two receivers. C_2 is usually provided in parts per million (ppm). Therefore, given SD_{A1} and SD_{A2}, the expected 2D horizontal closure error SD_A is according to the following equation:

$$\boldsymbol{SD_A} = \sqrt{\boldsymbol{SD_{1A}}^2 + \boldsymbol{SD_{2A}}^2} \qquad \text{Equation 12-2}$$

If the true 2D closure error vector is less than the standard deviation ($SD_A \times 2.45$), the GNSS observations are consistent with the manufacturer's specifications at a 95 % CL. Once the observations pass this test, the coordinates of point A can be averaged with coordinates 1A and 2A. A simple arithmetic average is often sufficient. However, if the lengths of the base lines appear to be substantially different, it is appropriate to use a weighted average according to the distances of the two reference points.

A simpler approach is to consider that the coordinates of the reference points are free from errors. Then the SD_A of the mean coordinates of point A can be calculated as follows:

$$SD_A = \sqrt{\frac{{SD_{1A}}^2 + {SD_{2A}}^2}{2^2}}$$ Equation 12-3

In the same way, the closure error test can be performed on all surveyed points and calculate the standard deviation of the mean of the resulting coordinates. To test the extent to which the survey matches the customer's requirements, the standard deviation of a vector between the average coordinates of the new points B and C can be calculated as follows:

$$SD_{BC} = \sqrt{{SD_B}^2 + {SD_C}^2}$$ Equation 12-4

In order to achieve the requirements (at 95 % CL), the value of (SD_{BC} × 2.45) must be less than the value specified by the customer.

It should also be noted that numerous large closure errors in surveyed points are an indication of poor coordinates at one or both of the points used as a reference point, or these points may have been damaged. Isolated large closure errors may indicate a weakness in the baseline solution.

12.3.5. Survey Design Development

If the results obtained with the closure error analysis do not meet the required specifications, there are several options for improving the survey design:

1. Higher precision techniques, e.g., static, fast static, can be used for long base lines. For example, this option reduces the SD_{1A} and SD_{2A} in the previous example.
2. A reference point closer to the project area can be employed. This also reduces SD_{1A} and SD_{2A} in the previous example.
3. Observations can be added from a third reference point. Depending on the length of the new baseline, this leads to improving the standard deviation of the mean coordinates of the surveyed point, reducing thus the value of the standard deviation.
4. Independent baseline measurements can be added in one of the reference points, preferably from the point closest (point *S*2 in Figure 12-1), giving the same effect as the previous option.
5. Other independent observations could be added by measuring directly between the new points, e.g., A and B. In this case, least square adjustment should be used instead of the aforementioned simplified closure analysis method.

Making the right choice varies from project to project according to logistic factors. In the previous example, option 4 may be the most realistic. In any case, if the new points are not on the same line and are evenly spread over the project area, many of the new points may not meet the required specifications due to the long baseline distances. In this case, it is advisable to intensify the reference points (Option No. 2) rather than to increase the additional observations (Options 3 and 4). It can also be said that if direct measurements are required to augment additional observations (Option 5), it is recommended to use the fast static method instead of the RTK method.

12.4.Reporting

12.4.1. Coordinate Form

All geodetic coordinates in the Kingdom are referenced to KSA-GRF17 and KSA-VRF14, with the aim of the KSA-GEOID21. Transformations from WGS84, on which GNSS solutions are provided, to the aforementioned systems are performed with the well-known 7-parameters (Helmert) transformation (Section 2.8). Ellipsoidal heights are turned into orthometric with the aim of the respective geoid model, KSA-GEOID21. Details on the transformation parameters are given in (GASGI, 2022) on which the interested reader is cited.

12.4.2. Survey Reports

A written report on the project must be prepared and signed by the person responsible for the geodetic survey. The final report shall be provided as evidence of successful completion at the end of the geodetic survey work. The report shall be prepared according to Section 4.4 and Chapter 6 of this Volume and include at least the following elements (MoMRAH, 2005):

1. A detailed description of the project summarizing the project's conditions, objectives, implementation methods and achievements.
2. The survey plan, the equipment employed, satellites used, and occupation times.
3. A description of the implemented data processing method, with an explanation of the programs and versions used, and techniques followed including their parameters, error modeling, ephemerides used etc.
4. A detailed summary and analysis of the least squares control work including both minimally and fully constrained adjustment: a list of observations and constraints included in the solution, a list of typical and absolute residuals, differences in weights, coordinates relative reliability, coordinate expected errors at a 95 % CL, etc.
5. Determining any data or solutions that were excluded from the network, with an explanation of the reasons for their rejection.
6. A map of the new project points and used control points shall be attached to the report at an appropriate scale. The report must also include a description for each mark.

12.5.Results Handling

Preparing an archive of survey observations and control data is the last step in completing the survey work. The archive has to be prepared according to Section 4.4 and Chapter 6 of this Volume, and include at least the following (MoMRAH, 2005):

1. The original field books.
2. Basic account documents.
3. The original receiver observations.
4. A report about calculated positions, estimated accuracies and residuals.

When archiving GNSS data, both raw and output data resulting from baseline processing must be taken into account. For raw GNSS data it is recommended to use the RINEX FORMAT. The results of baseline processing, can be exported from employed software’s project and archived. Alternatively, in case an independent 3d least squares adjustment software has been used, the original input file can be archived.

Notes on planning and survey procedures as well as field notes, should include the following elements of GNSS surveying:

1. Antenna location and eccentricity: This is the case where the antenna is not installed above the dot mark. A description should be given of the mark on which the antenna is centered, and the 3D eccentric displacement values - from the eccentricity mark to the point mark - used to correct the coordinate calculation.
2. The authority supervising the survey: the authority responsible for supervising the GNSS survey must be mentioned.
3. Receiver type: The type and serial number of the receiver and, if possible, the antenna type and serial number must be recorded.
4. Occupation time: The occupation date and time must be mentioned in UTC for the start and end period of the observing cycle; which are the times when the receiver starts and finishes recording satellite signals.
5. Frequencies used: All types of frequencies that were used have to be specified (L1/L2).
6. The satellites that have been observed: All satellites observed during the occupation time have to be recorded using a pseudorandom numbering system.
7. Describing control points: A full description of the monument/mark must be made, including the following:
    a. Stability.
    b. Type of monument/mark, including how it was built.
    c. Location relative to the surface of the earth and facilities, etc.
8. Additional Information: Any other information that may be of assistance should be given. For example, the height of the mark above the ground, the elevation mask angle for baseline processing, etc.

13. Aerial Surveying

13.1. Introduction

Photogrammetry is utilizing photographs to extract reliable three-dimensional metric information. The precision expected for any given hardware/software system is mainly dependent on camera to object distance. The next most important factor is georeferencing accuracy. This is accomplished either by Ground Control Points (GCP), or a GNSS enabled camera platform system, which is the most common case nowadays. Camera is the measuring instrument in photogrammetry, thus camera quality is critical to precision expectancy. The main advantage of photogrammetry is its ability to deliver any level of precision and accuracy with proper planning for photo and control acquisition.

The main disadvantage is that it can only measure what is recorded in photos. Therefore, structures and ground under vegetation or trees cannot be properly measured.

Photos may be captured from several platforms including satellites, manned, or unmanned aircrafts, the main difference being the area covered per time unit or flight endurance. Camera quality mainly refers to camera rigidity and secondary to lens and sensor quality, therefore photogrammetric cameras are heavy. Such cameras cannot be carried by small, unmanned vehicles and can only be mounted on manned aircrafts.

Photogrammetric deliverables can be vector maps, DSM, point clouds, contours, and orthophotomosaics. Digital orthophotomosaics are excellent background for any existing vector information. Creation of orthophotomosaics mandates the existence of DSM. The DSM can be created directly from aerial photos or from a LiDAR aerial survey. In case aerial photos are used then only DSM can be created meaning that the canopy of trees and vegetation will be recorded as height information, which may be troublesome for some applications, include volume calculations. This is the main difference between photogrammetry and LiDAR survey, since the latter may produce either DSM or DTM surface.

Aerial Surveys are used for cadastral mapping, topographic survey of large areas or preliminary design of roads and construction projects. Nadiral photography is used for topographic mapping, while the use of additional oblique photos should reduce point cloud noise, improve Z precision, and produce more points in vertical surfaces in cliffs and manmade structures such as buildings in urban areas. Nowadays processing of photo blocks is almost fully automated, exploiting SfM and MVS techniques. Stereoplotting, vectorization directly on stereo-pairs, is only used when precise vectorization of certain man-made features or land boundaries is expected. Automation also facilitates the rapid processing of huge numbers of aerial photos into DSM and orthophotomosaics products, which nowadays tend to fully replace traditional vector plots.

Manned or unmanned flights can be used for data acquisition, with the latter rapidly replacing the former. Modern mapping aircrafts are equipped with GNSS antennas able to provide either RTK or PPK precise positioning data for each captured photo. This ability reduces or even eliminates the need for GCP, helping to reduce overall time from flight commissioning to product delivery.

Aerial Surveys are an ideal tool for cadastre, topographic, terrain, planning, design, monitoring of linear infrastructures, such as roads, power lines, etc., or large construction sites.

Orthophotomosaics provides a detailed visual record of the current situation but need basic interpretation skills from the user. At the same time is an excellent documentation archive for monitoring construction progress and accepted as evidence in a court of law.

13.2.LiDAR Surveys

LiDAR (Light Detection and Ranging) utilizes a very rapid beam to scan its environment. As a principal LIDAR emits a laser beam, which bounces back to objects and returns to the instrument's receiver. Consequently, time is measured, and the distance from the object is calculated. If the position and orientation of the instrument is known, then the 3D corrdinates of the object can be calculated. LiDAR generates an enormous number of 3D object points in the form of a point cloud.

The main advantage of LiDAR is wavefront analysis, which means that for each emitted ray, multiple returns can be recorded. That way, both top of vegetation canopy and ground underneath can be recorded in a single pass.

The aerial platform accommodating the LiDAR must be equipped with high accuracy GNSS antennas and geodetic grade IMU, because the position of each ground point is related to the exact location and orientation of the LiDAR unit. By default, LiDARs can only record 3D points without any color information, other than the reflected power of the signal. This is considered its main disadvantage. Modern airborne platforms usually carry a separate camera to record photographs and color information for the recorded point cloud. Usually, these photos can also be used for photogrammetric processing and creation of orthophotomosaics, although flight can only be optimized either for photogrammetric or LiDAR acquisition; not both at the same time due to contradicting flight requirements.

Until now, LiDARs were heavy devices and could only be mounted on manned aircrafts. Modern LiDARs are lighter and more compact, hence some types can be carried from unmanned platforms, at the expense of area coverage, range, and flight height. Nevertheless, they are considered a suitable alternative for manned aircrafts, especially for small areas or corridor surveys such as those in road design and construction. In addition, the advantage of LiDARs in recording ground beneath vegetation is important for precise volume calculation. LiDAR aerial surveys utilizing a green laser can obtain bathymetric data up to 50 meters depth depending on water visibility.

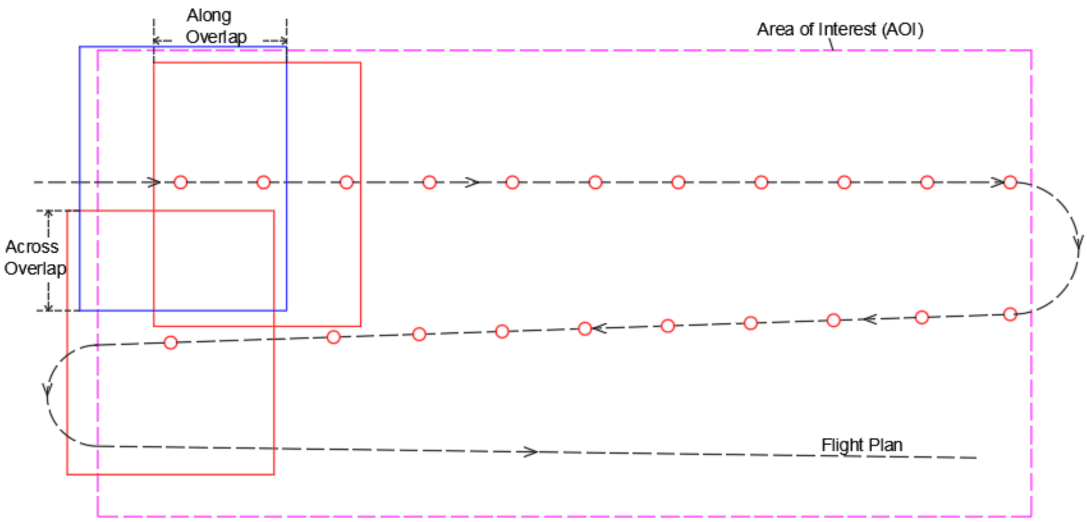


Figure 13-1 Typical Flight Pattern over Area of Interest

LiDAR aerial surveys can be planned in a way to fulfill any requirements in term of point density and accuracy, depending on flying height and LiDAR characteristics.

13.3.Aerial Photography

Principles of photogrammetry and photogrammetric processing are not affected by the platform used for photo acquisition. Therefore, in concept, processing is indifferent if photos were captured by unmanned (drone), manned aircraft or satellite. In the latter case, there are some chances in photogrammetric processing algorithms, mainly because of the image formation process, which differs from the central projection. In between manned and unmanned aircrafts there are only slight differences.

The most crucial factors when ordering or planning a flight acquisition are area extent, terrain morphology, flying height, camera/lens specifications and existence of onboard GNSS for direct georeferencing. Other factors affecting the flight are weather conditions and flight regulations. Camera lens and flight height are the most important factors affecting precision. Photo scale (Equation 13-1) and ground pixel size are directly affected by the lens and flight height. Ground pixel size is directly related to overall project accuracy and requirements. Photo scale is related to final map scale, but they must not be mistaken for each other.

$$\frac{1}{k} = \frac{focal\ length}{flying\ height} \qquad \text{Equation 13-1}$$

Photographic acquisition is performed in lines (Figure 13-1). All mapping aircrafts have dedicated software for flight planning. The pilot needs to setup basic parameters and then the flight is automatically calculated and uploaded to the aircraft’s auto pilot. These parameters, along with their explanation and importance are shown in Table 13-1.

Table 13-1 Flight Parameters for Both Manned and Unmanned Flights

<table>
<tr><th colspan="2">Flight Parameter</th><th>Notes</th></tr>
<tr><td colspan="3">Obligatory</td></tr>
<tr><td>Area Of Interest (AOI)</td><td>Closed polygon</td><td></td></tr>
<tr><td>Flying height
Or Ground Pixel Size</td><td>m</td><td>Defines precision and detail of the final map. These values are linked, and whichever is set affects the other one.
Whichever is set, the flight planning software displays the other one.
Not both can be set for a given camera/lens system.</td></tr>
<tr><td>Along or forward overlap</td><td>60 - 80 %
Typical 80 %</td><td>The more it is, the more photos are taken, more flight time.
60 % is the minimum and recommended only for flat areas. It increases for more morphologically intense areas, i.e., for urban areas shall be set in 80 %</td></tr>
<tr><td>Across or lateral overlap</td><td>25 - 80 %
Typical 60 %</td><td>The more it is, the more photos are taken, more flight time
60 % is the minimum and recommended only for flat areas. It increases for more morphologically intense areas, i.e., for urban areas should be set in 80 %</td></tr>
<tr><td colspan="3">Optional / Other to Consider</td></tr>
<tr><td>Flight direction</td><td colspan="2">Usually set automatically so that to minimize the number of photos. It is affected by the shape of AOI.</td></tr>
<tr><td>Terrain following</td><td colspan="2">It requires an external source of DSM information, or on-board sensors. Helps in maintaining a constant photo scale across the block. Important when there is significant height difference within the AOI.
Special caution must be exercised in unmanned flights, since unrecorded or strong surface undulations (cliffs, gorges, dingles) might not provide enough warning for the aircraft to gain height. Safety first</td></tr>
<tr><td>Cross flight lines</td><td colspan="2">Recommended only if there are manmade structures or urban canyons on the AOI.</td></tr>
<tr><td>Oblique photos</td><td colspan="2">Not recommended for smooth areas. Only for dense urban areas or areas with strong terrain variations</td></tr>
</table>

Flight Parameter	Notes
Flight time (unmanned flights, only)	Unmanned aircraft flight time is usually overestimated. Safety first

Photographs are acquired sequentially with at least 60 % along the flight line. When the aircraft completes the line, it turns around and acquires photos in a parallel line with at least 25 % overlap with the previous one. The flight continues in a similar manner until all the area is covered (Figure 13-1). Flight along and across overlaps are crucial for accuracy. Typical overlaps are 80 - 60 % along-across overlap but may vary depending on the characteristics of the covered area and the requirements of the aerial survey. The sparse overlap is used for flat terrain and agricultural areas, where as the dense one is for complex urban scenes.

The main advantages of aircraft platforms against satellite imagery are the increased spatial analysis and accuracy. They also excel in detailed 3D modelling of structures and terrain. The compliance of each platform for each map scale is provided in Table 13-2 and the suggested ones are shown in Table 13-9.

Table 13-2 Map Scales and Horizontal and Vertical Compliance for Orthophotomaps

Map Scale	Horizontal Accuracy			Vertical Accuracy Contour			
	Unmanned	Manned	Satellite	Unmanned	Manned	Satellite	LiDAR
1:500	✓[2]	✓		✓[2]	✓		✓[2,3]
1:1,000	✓	✓[2]		✓[2]	✓[2]		✓[2]
1:2,000	✓[1]	✓[2]		✓	✓[2]		✓[2]
1:5,000	✓[1]	✓[2]	✓	✓[1]	✓[2]	✓	✓[2]
1:10,000	✓[1]	✓	✓[2]	✓[1]	✓[2]	✓[2]	✓[1]
1:20,000	✓[1]	✓[1]	✓[2]	✓[1]	✓[1]	✓[2]	✓[1]

NOTES:

1. Not efficient, hence not recommended.
2. Best practice.
3. Only in unmanned platform and low flight.

13.3.1. Manned Platforms

Manned aircrafts are the most versatile platforms and can accommodate any kind of flight requirements in terms of flying height, map scale and duration. They are usually equipped with photogrammetric cameras of high quality and GNSS antennas for direct georeferencing. Photogrammetric cameras (such as Leica DMC III, Vexcel Ultracam Eagle, Phase One PAS, IGI DigiCam) are pre calibrated, with stable geometry and high-end lenses. Their sensors capture 4-band (blue, green, red and infra-red) photographs. The main advantage over unmanned platforms is their efficiency as they can fly more time and cover any given area quicker.

The main disadvantage of mount platforms is their cost and preparation time. Such flights should be planned many months ahead depending on the availability of the aircraft. Once commissioned, the flight company will take care of any necessary flight permissions.

13.3.2. Unmanned Platforms (UAV, Drones)

When smaller areas are to be mapped, manned flights are not efficient as the cost and deployment time are hefty. In such a case, the use of unmanned aircraft has significant advantages as they are affordable, flights can be planned quicker, and more companies can be found. Their main disadvantages are the fact that they cannot lift a lot of weight and the flying duration and range are limited, hence they can cover a smaller area per flight. Although the latter can be compensated by more flights, the fact that it cannot carry a large photogrammetric camera poses certain limitations in image quality. Recent improvements of unmanned platforms have enabled them to can carry much better cameras, are GNSS enabled, have more than 90 minutes flight duration and may even carry multispectral cameras or LiDARs. Therefore at least in terms of technology they are like manned platforms, although usually payloads are of slightly inferior quality.

Unmanned aircrafts' flying height limitations means they cannot fulfill all requirements in map scale, but on the other hand they can achieve smaller ground pixel size with increased detail. The increased detail does not necessarily mean increased precision because the cameras on board unmanned platforms are of inferior quality, which affects the final accuracy and precision.

Table 13-3 Advantages and Disadvantages of each UAV Type

	Pros	Cons	Typical Uses
Multi-Rotor	Accessibility Affordable Ease of use VTOL and hover flight Good camera control Can operate in a confined area. Safety (6-8-10 copters)	Short flight times Small payload capacity Safety (4 copters)	Aerial Photography and Video Aerial 3D modelling of monuments/sites LiDAR Small site mapping Renewable energy facilities inspection < 10 mm GSD

	Pros	Cons	Typical Uses
Fixed-Wing	Long endurance Large area coverage Fast flight speed Safety	Launch and recovery needs a lot of space. No VTOL/hover capability Harder to fly, more training needed. Minimum flying height might be limiting for some applications	Aerial Mapping Pipeline and Power line inspection Security Large construction monitoring Open mine volume monitoring > 15 mm GSD
Single-Rotor	VTOL and hover flight Long endurance (petrol powered) Heavier payload capability	Safety Harder to fly, more training needed. Expensive	Aerial mapping Security LIDAR
Fixed-Wing Hybrid	VTOL and long-endurance flight Safety	Not perfect at hovering Limited vendors at the moment	Drone Delivery Aerial mapping LiDAR

13.3.2.1. Main types

There are three main categories of UAVs, each one with unique flight characteristics. The main differences are flight duration, range, weigh capacity. These factors affect area covered within a single flight, sensors on board, takeoff and landing zone and overall efficiency. These factors are critical for selecting and planning flight missions.

It should be noted that although the platform (UAV) and sensors onboard are not related to each other, most vendors are now offering integrated solutions. Hence, the sensors and flight characteristics are merged in the specifications, providing a unique mapping solution. Advantages and disadvantages for each UAV type are summarized in Table 13-3.

13.3.2.2. Unmanned vs manned flights

The main advantages of unmanned surveys when compared to manned ones are:

- Cost savings.
- Reduced time between order to flight and flight execution, since logistics and planning are much simpler.
- Better (smaller) GSD, hence more detailed photos.
- Versatility of flight planning.
- On board GNSS antennas on UAVs are usually calibrated for lever-arm displacement among camera projection center and antenna center, ultimately providing directly the

position of the photo. This is not usually the case on manned aircrafts where the lever-arm needs to be estimated as unknown in the least squares bundle adjustment.

The main disadvantages are:

- Smaller sensors, hence smaller coverage per photo, leading to more photos and exponentially more processing time for a given area.
- Most modern UAVs are equipped with suboptimal cameras, hence although the GSD is better to manned flights with calibrated photogrammetric cameras, the results in terms of mapping accuracy are similar.
- The use of uncalibrated cameras in UAVs, require use of self-calibration during Aerial Triangulation (Bundle Adjustment). This fact increases the number of ground control points, hence more field time and increased cost.
- Several flights may be needed to cover the specified area, due to limitations in UAV flight time.
- Legal regulations may not allow flying over specific areas.

13.3.3. Factors to be Assessed

When considering commissioning or performing UAV flights for mapping the main factors to taken into consideration are:

- Type of UAV for range, area coverage and takeoff and landing position
    - Multi rotor; less area coverage per flight (~1 km^2), small takeoff and landing area, better wind resistance and stability, better for small areas and detailed mapping.
    - Fixed wing; better area coverage (> 2 km^2), better for corridor mapping and large areas, need proper area for takeoff and landing.
    - Hybrid; combine the advantage of both types, may take off and land in small areas like a multirotor, but can cover large areas in a single flight.
- Camera
    - The photo scale or equivalently the GSD are the most critical factor in any photo acquisition project and they are defined by the combination of focal length (c) and flying height (H) (Equation 13-1
      Equation 13-1).
    - Camera sensor (complementary metal-oxide semiconductor - CMOS) physical area. The larger it is, the larger the photo coverage resulting in fewer photos for a given area. One-inch sensors or better should be used. Full frame sensors are becoming the norm for photogrammetric flight acquisition while large format sensors (Phase One camera family) are already integrated in high end mapping UAVs.
    - Camera calibration and lens quality are critical factor for the processing. As a rule of thumb, the larger the camera sensor the better the camera and its lenses.

GNSS equipped UAVs are becoming standard for mapping projects because they allow for processing without any ground control points. Since the acquisition of ground control points in corridor surveys are a tedious task, both in terms of time and cost, using a GNSS (RTK or PPK) UAV, with a GNSS base station or Network support, is highly suggested.

Other causes for consideration are:

- Legislation for unmanned flights, permissions, operator's license, insurance, etc.
- Permission for Beyond Visual Line-of-Sight (BVLOS) flights, which will significantly increase coverage and efficiency.
- Weather conditions affecting the flight, such as wind speed and direction.
- Battery health.
- Peculiarities of the aircraft and familiarity with them.

Safety checklist and safety considerations are always the first and last concern in any flight.

13.3.4. Satellite Imagery

The main difference of satellite images in comparison to aircraft platforms, is the image formation process as their corresponding geometry does not comply to central projection. Most satellite cameras operate in pushbroom mode, which means that their shutter is continuous and forms the image one line at a time. In practice this means that processing software must have different algorithms to process such images.

Currently the best commercially available ground pixel size by satellite is 0.3 meters. Some satellites provide the option of stereoscopic image acquisition to extract DSM. Given the peculiarities in image formation process, their large flying height, and the distance among their paths, their DSM height accuracy is inferior to what their ground pixel size suggests.

Therefore, although DSM can be extracted by satellite images, it is usually avoided, and other sources of terrain information are preferred. In fact, in most cases monoscopic coverage of satellite images is combined with DSM from another source (LiDAR, contours, existing DSM), for ortho rectification.

13.4. Data Processing from Aerial Surveys

Combined processing or aerial surveys and LiDAR follow the methodology shown in Figure 13-2. For orthophotomosaic production, an existing DSM is needed. This DSM can be created either by aerial photography or by LiDAR data.

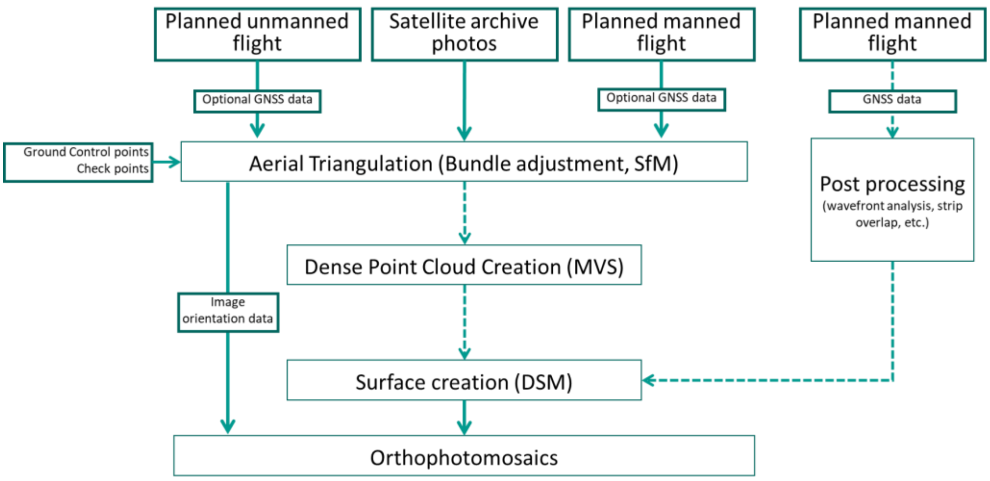


Figure 13-2 Aerial Survey Processing Pipeline

13.4.1. Aerial Triangulation

Aerial triangulation is the most important process in photogrammetry. It dictates the final precision and accuracy of the photo block, hence it is important for the operator to adjust critical parameters and have the ability to access internal quality measures.

Most modern photogrammetric software relies on automated Structure from Motion (Sfm) algorithms to align photos and perform Least Squares adjustment (Bundle Adjustment). There are several commercial options for photogrammetric software, with different levels of automation and user involvement. Some of them perform like black boxes, where photos are imported and the final orthophoto mosaic is exported with minimum user participation. Such software solutions should be avoided since they do not provide flexibility nor extensive quality measures. An overview of the aerial triangulation process, input, output and types of points are shown in Figure 13-3. There are three types of points in this process: tie points, control points, check points (Table 13-4).

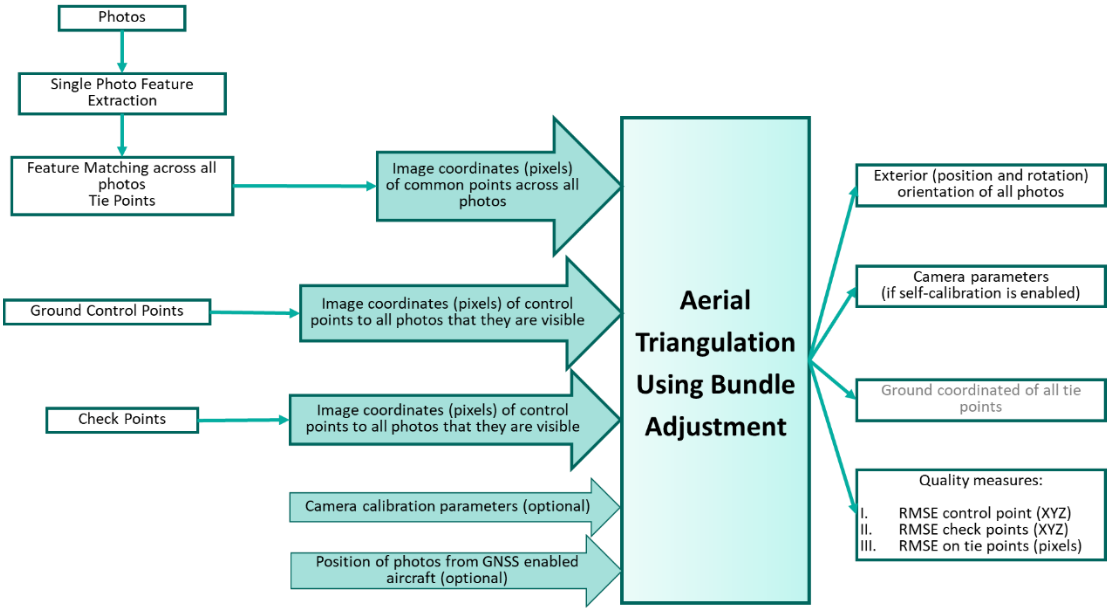


Figure 13-3 Aerial Triangulation Input and Output

Tie points are common points among photos, ensuring connection of photos within the block. They are responsible for relatively aligning all photos in 3D space. Their distribution, multiplicity across as many photos as possible and overall quality will affect rigidity of the block.

Ground control points are required for the georeferencing of the block. They may be either pre signalised or physical points. In the latter case they must be well defined points, visible and recognisable at project's photo scale. In both cases, GCPs size is related to ground pixel size, so that they are easily detected and well defined for fine measurement on the photos. Their XYZ coordinates are known since they are measured in field with RTK GPS (expected accuracy better than 0.03 m horizontally and 0.05 m vertically). These points are then detected and measured across as many photos of the block as possible.

Table 13-4 Types of Points Involved in Aerial Triangulation

Type	Measured in field (known XYZ)	Measured on Photos	Used in BA	Function- Role
Tie		Automatically	✓	Ensure alignment and connection among photos of the block. Critical for precision
Control	✓	Manually	✓	Ensure georeferencing with KSA-GRF17, KSA-GRV14. Critical for accuracy
Check	✓	Manually		Validate precision and accuracy of the result

For the evaluation of the results of the aerial triangulation checkpoints must be used. These points are also measured in field but not used during Least Squares Adjustment (Bundle Adjustment - BA), hence do not participate in the calculations.

One important aspect in aerial triangulation is the relation among tie points and ground control points' weights, which is defined in BA parameters. In other words, by favouring one over the other the results of the triangulation might differ significantly in essence, in figures and in quality measures (RMSE). As mentioned before, tie points are responsible for the shape and rigidity of the block while control points are responsible for referencing the block to the ground. There are two approaches:

- By benefiting the ground control points (i.e., declaring GCP accuracy of 0.01 m in X, Y, Z) block shape is forced to deform to meet ground control point positions. This will decrease RMSE on GCP (ground units) and increase the RMSE on tie points (image units). Consequently, although the RMSE in GCP will suggest a good aerial triangulation, when measuring a random point in the block, its position will be much worse than the RMSE on GCP will suggest. This is the reason, one should check the RMSE on tie points and use check points, as the RMSE on check points will reveal the weakness of the solution.
- By benefitting tie point accuracy (i.e., declaring measuring accuracy on photos 0.25 pixels), the internal geometry of the block will be strengthened the shape of the block will become very rigid but it will not satisfy ground point positions. As an effect the RMSE on GCP will be increased suggesting a bad solution, although that the internal precision of the block will be good.

To overcome such inconsistencies, weights should be set with realistic values, so that quality measures represent the quality of the adjustment:

- Ground control point X, Y, Z accuracy as measured in field. It is dictated by the measuring method (GNSS, RTK, PPK, single frequency GPS, theodolite, etc) and equipment specifications.

- Ground control point accuracy as measured on photos, in image units. It is dictated by the distinctness of the control points from their background and operators experience. It may be estimated and usually varies between 0.5 - 2 pixels.
- Tie point accuracy in image units. This depends on the feature extraction method of the software and the geometric quality of the camera. It may be estimated with values starting from 1 pixel for photogrammetric cameras and subpixel feature extractors, up to 3 pixels, for non photogrammetric cameras and standard feature extractors.

Best software will allow for setting all these parameters and sometimes even allow for setting separately values for planimetric and vertical accuracy of GCPs, or even different values on each GCP to compensate if they were measured with different methods/equipment.

After BA the position of each photo of the block in 3D space is calculated with high precision, this being the main outcome of the aerial triangulation. During BA other unknowns can be calculated as well, such as camera calibration, if non photogrammetric cameras were used. The process of using a camera with unknown parameters, which are then calculated during BA, is called self-calibration. Other unknowns that can be calculated during BA. are parameters of the 3D vector between camera projection centre and GPS antenna on board the aircraft.

A camera self-calibration should be performed:

- When camera parameters are unknown.
- All times when a non-photogrammetric camera is used, even if camera parameters are known from a previous project. These cameras are not rigid enough to sustain previous calibration parameters.
- When we wish to decrease RMSE even if a calibrated photogrammetric camera has been used.
- When parameters of a photogrammetric camera are known, but it is suspected that the camera suffered deformation (during transport, because of extreme heat/cold conditions, etc).
- When the software does not support input of a calibrated camera, hence all cameras are to be calibrated anyway.

The camera parameters that shall be set as unknowns are shown in Table 13-5.

Table 13-5 Camera Calibration Parameters

Focal length	**Principal point**	**Radial lens distortion**	**Tangential lens distortion**	**Non orthogonality**	**Rolling shutter**	**When**
✓	✓	✓				Photogrammetric camera
✓	✓	✓	✓			Not photogrammetric, uncalibrated camera
✓	✓	✓	✓	✓		Low quality camera

Focal length	Principal point	Radial lens distortion	Tangential lens distortion	Non orthogonality	Rolling shutter	When
✓	✓	✓	✓	✓	✓	Video frames, Low quality camera without mechanical, nor global shutter
Focal length: f Principal point parameters: x_o, y_o Radial lens distortion parameters: k_1, k_2, k_3 Tangential lens distortion parameters: p_1, p_2 Non orthogonality parameters: b_1, b_2						

The georeferencing process aligns the block of photos to the KSA-GRF17 and KSA-GRV14. Two methods can be used. The first one uses the ground control points to correlate the aerial block with the ground. The second one uses the GNSS recorded geographic coordinates of each photocenter to correlate the block with the ground. On the latter one no ground control points may be used. In any case RMSE on check points shall be calculated as a mean to avoid gross errors or as an estimate of block accuracy.

13.4.1.1. Tie points

Contemporary aerial triangulation software use SfM methodology to automatically extract a huge number of tie points across the block. Filtering and manual deletion of gross error of tie points should be performed before BA, but not all software provides such functionality.

Tie points are unsignalized but well identifiable ground points in photos. Each one should be measured in as many photos as possible. When a point is measured in many photos it increases rigidity of the block. There are no other requirements for tie points other than being clearly visible in as many photos as possible photos. Automated algorithms are very good in identifying a huge number of points per photo and matching them across images. Nevertheless, they are prone to mismatches, and they identify each tie point on three photos on average, which is suboptimal (at a typical 60 - 80 % overlaps flight, the average is 7 photos). The number of extracted features per photo varies depending on photo content. Photo resolution can be used as a measure of indented extracted features, which shall not exceed 0.2 % of total pixel count evenly distributed over the photo.

While automated algorithms are very effective for small blocks, when the number of photos increases then the processing time increases exponentially, unless some apriori knowledge of the photo layout is used. Photo resolution also increases processing load. Therefore, sometimes an initial alignment may take place, using low resolution photos (by subsampling the original photos) to obtain a rough alignment of the photo layout. This information can be used on the second alignment with full resolution photos as apriori knowledge of their position to minimise search of tie points only to neighbouring photos.

It should be noted that in areas with poor photo texture the algorithms might fail to recognise and match tie points.

The quality measure which is related to tie point quality is the RMSE of tie points, expressed in image units, usually pixels. When using a calibrated camera (photogrammetric camera) this value is at the order of 0.7 pixels. When using uncalibrated cameras, it may be as high as 1.5 pixels. This value is very sensitive in mismatches, and this is why filtering of tie points should take place before final bundle adjustment. If filtering of tie points is supported by the software, then it should be taking advantage of. Indices that could be used to filter tie points are:

- Epipolar constraint – remove points with residuals larger than 1 pixel for calibrated cameras, or larger than 2 for a poor-quality uncalibrated camera
- Intersection angles lower those 10 degrees is recommended and intersection degrees lower than 5 degrees shall be avoided.

Filtering of erroneous tie points should take place using relaxation method. For example, if epipolar residuals are as high as four pixels in the initial run, then point larger than 2.5 pixels should be removed. Then the bundle adjustment should take place to calculate residuals and then remove tie points with residual larger than one pixel. No more than 5 % of the total type point count shall be removed in a single step. Bundle adjustment and recalculation of residuals should take place in between each removal step.

13.4.1.2. Ground control points requirements

Distribution of ground control points is crucial to accuracy and precision of the block. The distribution varies according to block shape, overlaps, and use of calibrated or uncalibrated camera. Four control points at the corners of the block must be placed. As a rule of thumb, on a block with typical 80 - 60 % overlaps, ground control points should be placed:

- At the four corners of the block.
- At the first and the last photo of each other flight line.
- At the first and last line of the block, one ground control point should be placed every 10th photo.
- Additional control points should be evenly placed in the middle of the block, roughly one every 40 photos if block has 100 or fewer photos, one every 50 photos if block has 100 - 500 photos and one every 70 photos for larger blocks.

In case an uncalibrated camera is used then the need for control points is doubled in the middle of the block. Control points shall be placed on the lower and highest areas of the AOI.

Corridor surveys, such as those in road mapping, are much more vulnerable in height accuracy. For that reason, ground control points should be placed:

- At first and last photo of each flight line two control points should be positioned
- Additionally, one control point every fifth photo is necessary. The controls shall be placed alternatively at its side of the flight line.

As a general note, cost and delays of ground control points is considerable and should not be underestimated. This is the reason most mapping aircraft are equipped with GNSS antennas for direct georeferencing. For example, using an unmanned platform with uncalibrated cameras to cover a large area, with many small photos, will increase the number of necessary control points. If the area is big, then travelling time should also be considered. If the area mapped is barren, without roads, then travelling to defined positions in the wilderness will take more time, need of special vehicles and navigation assistance. All these issues are reflected to the final cost, sometimes rendering an initially attractive unmanned flight in failure.

As a rule of thumb, RMSE on ground control points shall be three times less than the RMSE of the final map requirements as set by Table 2-1 and Table 2-2.

In case of satellite images at least four control points shall be used per image, although more points would increase accuracy. In case of large overlaps and stereoscopic coverage, tie points may be used to increase precision.

Because of peculiarities of each project (area shape, terrain morphology, GCP distribution, block geometry, camera quality, sensor resolution, etc), GCPs' RMSE cannot be considered a reliable measure for block precision and accuracy estimation. This role is assigned to check points, which are not used during BA calculations, hence are unbased estimators.

Table 13-6 Map Scales, vs Horizontal Accuracy vs Ground Pixel Size of Original Photos

Map Scale	Horizontal Accuracy RMSE (m)	Maximum GSD of Original Photos (m)
1:500	0.13	0.11
1:1,000	0.25	0.20
1:2,000	0.50	0.41
1:5,000	1.25	1.02
1:10,000	2.50	2.04
1:20,000	5.00	4.08

13.4.1.3. Check point requirements

Aerial triangulation checkpoints should be evenly distributed over the block area. They should be no less than 4, and at least 1/4, of the ground control point quantity. In case the aerial triangulation was performed using GNSS photo centers, then the check points should be 1/4 of the equivalent ground control points should they have been used.

The RMSE on ground check points shall be below the RMSE of the final map requirements as set by Table 2-1 and Table 2-2.

In case of satellite images at least four check points should be used per image.

The maximum Ground Pixel Size of the images, to fulfil horizontal accuracy requirements for each map scale can be seen in Table 13-6.

13.4.2. Point Cloud and DSM Generation

Point cloud generation is based in multi view stereo (MVS) and it is completely automated in modern photogrammetric software. The density of the point cloud can be as high as one point for every picture of every photo, but such high density is unnecessary and requires more processing time. Since photogrammetric point clouds are noisy, any excessive number of points is being used internally by the software for noise removal. Density of terrain point shall be no less than 10 times the ground pixel size or 0.5 mm x scale factor of the final map, whichever returns more points. Other than density, there are only limited parameters to be set. Sometimes software may apply threshold for the minimum numbers of photos where the point is visible at. Although minimum might be set to 2 to increase the number of points, it is advisable to prefer a threshold of three photos in order to increase reliability of the remaining points. Maximum GSD of original photos per map scale are given in Table 13-7.

Table 13-7 Map Scales, vs Spot Elevation Vertical Accuracy vs Ground Pixel Size of Original Photos (70 % overlap, 100° lens assumption, with H/B ratio of 2)

Map Scale	Spot Elevation Vertical Accuracy (m)	Maximum GSD of Original Photos (m)
1:500	0.05	0.04
1:1,000	0.10	0.07
1:2,000	0.25	0.18
1:5,000	1.50	0.35
1:10,000	1.00	0.71
1:20,000	2.50	1.77

Visual inspection should take place before DSM generation to remove noise and gross errors. This can be done automatically by thresholding precision of extracted points if software supports such functions.

The accuracy of each point shall be compatible with the spot elevation accuracy of Table 2-2. Contour lines can be automatically created by the photogrammetric software. Otherwise, the point cloud or the DSM (in TIFF format) can be exported and used in GIS software where the final map production will take place. In any case, contour smoothing shall take place before final plotting (Chapter 6).

13.4.3. Orthophotomosaics

The creation of ortho photo mosaics is a fully automated process in modern photogrammetric software. Each georeferenced image (aircraft or satellite) is being orthorectified using a DSM. The source of DSM may be a LiDAR survey or created from photogrammetric processing of the photos of the block. Any combination whose photo source and DSM source fulfil respective accuracy requirements, is allowed.

Table 13-8 Map Scales, vs Ground Pixel Size of Orthophotomosaic and Recommended GSD of Original Photos

Map Scale	Horizontal Accuracy RMSE (m)	Ortho Photo Mosaic GSD (m)	Maximum GSD of Original Photos (m)
1:500	0.13	0.05	0.05
1:1,000	0.25	0.10	0.09
1:2,000	0.50	0.20	0.18
1:5,000	1.25	0.50	0.45
1:10,000	2.50	1.00	0.91
1:20,000	5.00	2.00	1.82

The final ground pixel size of the ortho photo mosaics is defined by the user but should correspond to ground pixel size of the original photos and by no means be smaller than the original ground pixel size. The final ortho photo mosaic may have at least 1.1 x the GSD of the original photography (Table 13-8). Since the orthophoto is differential rectification of the original photo, resampling on pixel values is performed. If software provides options for resampling method, then bicubic should be selected as it is the one retaining most information of the original photo.

The orthorectified photos of the block are merged into a seamless orthophoto mosaic. Seamlines are automatically created from the software and are invisible in the final product. During this process colour balancing takes place across adjustment orthophotos and across the block.

13.4.4. Vector Plots

Orthophoto mosaics are a great product for mapping continuous and smooth surfaces without such as terrain. This is not the case when vertical surfaces such as those in man-made objects (i.e., building walls), exist. There, the DSM cannot fully model such surfaces and artefacts appear such as edges in the final orthophoto (Figure 13-4). In such cases, using orthophotomosaic as backdrop to vectorize the outline of buildings is not recommended. The same holds for landmarks and other structures. For the orthophotomosaic to fulfil accuracy requirements for a particular map scale for such details, the orthophoto mosaic GSD of the Table 13-8 should be decreased by at least a factor of two. This renders the whole project inefficient, increasing the acquisition and processing costs.

Traditional stereo plotting vectorisation should be preferred when:

- Building and man-made features are needed.
- Land boundaries are needed.
- Spot heights and specific points must be recorded in highest accuracy.

Figure 13-4 Orthophoto from Building and Detail on the Edge

The photogrammetric process remains the same until after the aerial triangulation, when the results of BA are used to display user selected pair of photos in appropriate stereoscopic displays where the user may use a 3D mouse for precisely digitizing the edges of the buildings and other details as described by project requirements. The precision of stereplotting is equal to the RMSE of check points after aerial triangulation.

The stereo plotting does not exclude the creation of ortho photos as a backdrop for visualization of secondary details of the area.

Table 13-9 Map Scales, vs Ground Pixel Size of Original Photos and Most Appropriate Platform

Map Scale	Ground Pixel Size of Original Images	Unmanned Platform	Manned Platform	Satellite
1:500	0.04	✓	✓	
1:1,000	0.08	✓	✓	
1:2,000	0.19		✓	
1:5,000	0.39		✓	✓

Map Scale	Ground Pixel Size of Original Images	Unmanned Platform	Manned Platform	Satellite
1:10,000	0.78			✓
1:20,000	1.94			✓

13.5.Choosing Best Methodology for Each Scale

Having so much versatility it's not easy to choose the best methodology for each case and scenario. Nevertheless, the most efficient method for data acquisition becomes apparent when considering the ground pixel size of the original photography, out of which the most suitable platform can be detracted. Pivot Table 13-9 summarizes Table 13-6, Table 13-7 and Table 13-8 by retaining the smallest Ground Pixel size fulfilling all needs of horizontal, vertical accuracy and detail for the final map.

Having said that one must keep in mind that Lidar acquisition is the most accurate and precise for terrain extraction because of LiDAR's multiple return signals. LiDAR DSM can be combined with any imagery to generate orthophoto mosaics, provided both DSM and imagery fulfil the final map accuracy requirements.

It is also worth noting that although stereo pairs of satellite imagery can be used for DSM extraction, this is the least accurate method.

14. Laser Scanning

LiDAR (Vosselman & Maas, 2010), is a method using a laser to measure ranges (distance) by targeting an object and measuring the time light needs to travel towards the object and back. It is an active method, i.e., it provides its own illumination source, hence does not rely on external sources and can work at night as well as during the day. Its output is a low noise point cloud, colored by the intensity of the returned signal. LiDAR systems which incorporate calibrated cameras, can also return natural color information for each point.

LiDAR sensors may be static on tripods (terrestrial), airborne (Section 13.2) or mounted in cars' rooftops (Chapter 15). When LiDARs are mobile, they rely on several other sensors to determine their absolute position and orientation in space, to precisely locate object's points in 3D space. The accuracy of the points in the final point cloud is heavily affected by the accuracy of the supporting sensors. Such sensors are usually GNSS, IMU and more recently visual odometry (Visual Simultaneous Localization and Mapping (vSLAM)).

They are used when a digital 3D representation of an object or earth's surface is needed (Toth & Shan, 2009). Most common applications are in surveying, geodesy, cultural heritage recording, geology, geomorphology, seismology, forestry, atmospheric physics and laser altimetry. Most recently they are also used in the control and navigation of cars or autonomous unmanned vehicles (Jayaweera, et al., 2018).

A unique feature of LiDAR sensors is the ability to record multiple return signals over a single transmission. The ability of a LiDAR signal to penetrate vegetation and record an initial return from the canopy and a last one from the ground, allows for the vegetation to be removed during post-processing, or estimate vegetation height, used in aerial surveys (Chapter 15). Therefore, LiDAR may record DSM and DTM at the same time, with the restriction of man-made structures, which are inevitably recorded within the final point cloud. Reliable removal of man-made objects requires manual selection and removal, or extensive additional information, i.e., building footprints, breaklines, etc.

In this chapter we focus on static TLS used in road projects, being employed, but not limited, for the purpose of slopes survey and monitoring, building surveys etc.

14.1. Equipment

Regardless of the mounting point, LiDAR scanners are based on a laser emitter and a receiver. There are three main categories:

- Mechanical with rotating mirrors.
- Microelectromechanical (MEMS).
- Solid-state.

It should be stated that there is a price gap between the mechanical and the other scanners, with the former often costing hundreds of thousands of USD (at the time of this code) and the latter two categories are approximately ten times cheaper.

14.1.1. Mechanical

3D laser scanners have several rotating mirrors to rapidly direct the laser and collect thousands if not millions of points within a second. These expensive systems (Figure 14-1) are of high precision, reliable yet sensitive systems, and are intended for geospatial professionals. They are shock and vibration resistant, but they might require calibration in case of a tough shock or fall. Terrestrial laser scanners, used in stationary LiDAR surveys, fall into this category. They are able for approximately 360 ° x 270 ° scanning around the scanner, leaving only a small area around the tripod unscanned.

Leica Scan Station P30/P40 (top) and BLK360 (bottom). Size not in scale.

Source: https://leica-geosystems.com/

Riegl VQ-880-GH (top) for manned aerial platforms and VUX-120 (bottom) for UAV. Size not in scale

Source: http://www.riegl.com/

Teledyne Lynx HS600 mobile mapping system with GNSS, cameras and LiDARs

Source: https://www.teledyneoptech.com/

Figure 14-1 LiDARs with Mechanical Scanning

14.1.2. Microelectromechanical

These scanners are much smaller, lighter and portable scanners with low energy consumption. They still contain a small number of small moving parts, so they are still shock and vibration sensitive. Their scanning area is limited in a single plane or a limited field of view. They are mainly developed for autonomous navigation platforms and they are affordable, as they are mass produced for the robotics and automotive industry.

14.1.3. Solid State

They are often confused with MEMS scanners, due to similar size and cost. They rely on different technology and have no moving parts at all. They usually offer a smaller field of view, which is accepted in automotive industry if they are to scan objects in the moving direction in front of the vehicle.

Both MEMS and solid-state scanners, because of their affordability, low consumption and light weight, are being integrated into geospatial systems such as UAVs or similar mobile platforms (Chapters 13 and 15), providing a low-cost alternative to the mechanical ones. Their main limitation is their small field of view and a comfortable but small range in comparison to the mechanical ones, at about 200 m maximum.

14.1.4. Distance Measurement Methods

In order to assess the distance between sensors and surveyed objects, TLS utilizes two approaches, namely, Time-of-Flight (ToF) and phase-shift. Both types of rangefinders may absorb or reflect energy differently, depending on their specific design. These methods are presented in the following sections.

14.1.4.1. Time of flight scanners

ToF scanners send a pulse and wait for its return to measure distance from the object based on (Equation 14-1). ToF also known as pulse based) scanners are slow in speed, since they must wait for its pulse to return to the receiver and be measured. Their speed is a few thousand points per second, typically between 50 - 150 thousand points per second. On the other hand, they have extended range with a maximum up to 2 km or more. ToF provide the ability for waveform processing, which allows for internal processing capabilities to identify multiple returns or reflections of the same signal pulse. This provides multiple object detection, sometimes up to 15 returns per pulse. Therefore, the scanner can record both the return signal from vegetation leaves and the ground under it. Typical accuracy of the signal measurement is 2 - 6 mm.

$$\boldsymbol{Distance} = \frac{\boldsymbol{Speed\ of\ light \times Time\ of\ flight}}{\boldsymbol{2}} \qquad \text{Equation 14-1}$$

14.1.4.2. Phase shift scanners

The phase-shift laser scanners modulate the continuously emitted laser light into multiple phases and compare the phase shifts of the returned laser energy. Because of the need to continuously emit laser energy, their range is lower than a ToF scanner, but their speed is considerably higher, usually one million points per second or above. In phase-shift scanners a laser beam with modulated optical power is emitted and reflected off an object. The reflected light is then detected and compared with the emitted light to determine the phase shift and time of flight according to:

$$\boldsymbol{Time\ of\ flight} = \frac{\boldsymbol{Phase\ shift}}{\boldsymbol{2\pi \times Modulation\ Frequency}} \qquad \text{Equation 14-2}$$

Time of flight calculated by Equation 14-2, is transferred to Equation 14-1, to estimate the distance. Often multiple modulation frequencies are used to increase the accuracy of the time-of-flight estimation. Typical accuracy varies between 2 - 10 mm, while maximum range is limited to 300 m or less.

Density of points depends on settings and distance from the object. In terms of Signal -to-noise Ratio (SNR) the comparison of ToF and Phase-shift scanners depends on the application's specifics. They depend on several factors (distance, material, inclination angle, etc.), rather than the technology used. Nowadays, some manufacturers provide hybrid scanners using both technologies.

14.2. Procedure

Survey with TLS is divided in several static surveys from selected stations, in the same sense of a traverse with TPS. In fact, their distance measuring methodology is based on the equivalent distance measurement module of TPS. At each station, a large number of 3D points is collected and stored. Contemporary TLS are equipped with low grade cameras and can provide natural color information for each point in the point cloud, along with its reflected signal intensity.

14.2.1. Instrument Calibration

TLS equipment should be calibrated by repair and maintenance specialists according to the manufacturer's suggestions. While TLS are shock and vibration resistant, they might require calibration in case of a tough shock or fall. Requirements regarding instrument calibration posed in Section 4.2 for all topographic equipment, are also applied in TLS surveys.

14.2.2. Instrument Care

TLSs are regarded as fragile and sensitive devices since even a small shock can have a significant impact on how well they perform; repairing them back to their original state can be performed only by professional maintenance specialists. As a result, such devices should be handled carefully. Additionally, they should not be left exposed by any means, especially when operating near overcrowded areas such as highways, etc. Furthermore, the general guidelines should be followed as described in Section 4.2.

Instrument care actions and guidelines to avoid damaging the device, affecting the measurements quality and securing the uninterrupted operation of the device are similar to the ones employed in TPS surveys (Section 7.4), as listed below:

- Distraction or loss of attention could lead to device falling to the ground. The device should not be left unattended.
- Instrument should remain stable during scanning operations.
- Instrument should not be forced, as this may lead to it being damaged.
- Instrument can be moved mounted on the respective tripod, but caution is advised as sudden movements may damage it.
- The optical surfaces (such as scanning mirror, etc.) of the TLS instruments should be kept clean. Never rub off dust or debris off the optical surfaces as this will scratch the glass and may damage the instrument, instead, use compressed gas duster. TLS device shall be ensured of being switched off with its battery removed before cleaning it.

- The instrument should not be exposed to extremely low or high temperatures. Ideal conditions for the operation of the instrument are described by the manufacturer and should be followed. Furthermore, rapid temperature changes should be avoided when moving the instrument, giving time to the instrument to acclimatize.
- If the wind is severe enough to affect the stability of the instrument, the work should be stopped. Additionally, it is recommended to avoid exposing the instrument to direct sunlight for prolonged periods of time.
- The instrument should always be set up on its floor stand or tripod. Using the tripod specified for the scanning system guarantees maximum stability during scanning operations. Avoid setting up the instrument on the ground without floor stand or tripod.
- There is only one specific, designated position for the TLS device to be successfully placed inside its case. Damage to the device can be caused by forcing or pressing it inside the case. These actions should be avoided because of the delicate and sensitive nature of such devices.

According to international standards IEC 60825-1 and technical report IEC TR 60825-14 (2004-02) (IEC, 2004) TLS instruments are classified as laser class 1, class 2 and class 3R. Thus, they do not require:

- Laser safety officer involvement.
- Protective clothes.
- Eyewear and special warning signs.

14.2.3. Instrument Preparation

TLS Equipment should always be checked before each work day. Things to check:

- Instrument is functional and works as intended.
- All the possible accessories (tripod, protective case for secure transportation, spare batteries, charger, etc.) of the instrument are functional and prepared for use.
- Batteries of the instrument should be fully charged or at least be able to charge the batteries on the working site. It is strongly advised to carry one or two spare batteries for the instrument.
- Additional accessories that accommodate the use of these instruments, such as smartphones and tablets that connect with the device should also be fully charged and ready for use.
- Sufficient internal storage of the TLS device, regarding the project.

14.2.4. Instrument Setup

A point cloud with color and intensity information for each point makes up the raw measurements from a TLS at the scanner's reference system. Each point is categorized according to its return if wavefront ToF is employed. Compared to instrument operation of TPS as described in Section 7.5, TLS devices are easier to set up and use. TLS instruments do not require centering compared to TPS, but horizontal adjustment of the TLS instrument can be applied if the tripod offers that ability. After removing the TLS instrument from its case and setting it up on the tripod, the horizontal adjustment follows:

- Horizontal adjustment of the TLS should be carried out before each scan and can be achieved by viewing the level bubble or the inclinometer. Adjust the height of two tripod legs so that the bubble is close to center. If no level bubble is available, the horizontal adjustment can be performed approximately.
- Once the setup has been achieved surveyor can power on the TLS instrument in order to begin scanning. Simultaneously, user can view in real time the data recorded by the TLS device, as many TLS offer the option to be connected with smartphones/tablets and using the dedicated software (app) developed by the manufacturer. This option gives the user the ability to view and manage the data recorded by the LTS instrument in real time, on the site.
- The measuring duration may vary between one to several minutes depending on the TLS device, the density and type of the scan, etc. During the measuring process TLS shall remain stable; moving or touching it is prohibited.
- TLS devices scan in various fields of view depending on the TLS type. During each scan the user should remain unseen of the TLS device’s field of view or at least not stand too close to the instrument, in order for the scan to capture as much information about the surroundings as possible.
- After each scan is completed, the user may move the TLS device (mounted to the tripod) to the next scan location. In order for the scans to be successfully connected the scan data should achieve at least 60 % overlap (or according to the TLS manufacturer’s instructions) between scans.
- Highly reflective (mirrors, polished metal), absorbent (black) and translucent (clear glass) surfaces are unfavorable for scanning, so it is required caution when scanning such objects.
- Obstructions such as trees or columns will often require adjusting scanning positions to ensure all scanning data can be captured around the scanning area. Overlap beyond the obstruction between the scans is required.
- Other obstructions of small or bigger moving objects caused by unfavorable weather conditions such as tree leaves moving cause by windy weather etc. could influence the quality of the scans.
- Scanning such surfaces may result in misleading data acquisition or data loss that may affect the registration process. If necessary, the user can color, powder or tape these surfaces to limit the implications of scanning such surfaces.
- Low lighting does not affect the measurements since most TLS instruments have built - in self-illuminating technologies (e.g. flashlight etc.).

14.2.5. Scanning

During the scanning process the user should:

- Not move or touch the TLS instrument.
- Check that all the parameters of the scan, e.g., scan accuracy, density and color, are correctly selected and applied.
- Always check for sufficient internal storage of the TLS device.
- At least three to four scans (when applicable) should overlap in order for the point cloud registration to be as rampant and accurate as possible. Scan networks should have multiple scans overlapping and covering the same areas (Figure 14-2).

- Scanning order matters, because the registration process would be significantly simpler if the scans are performed in a logical order. It will be easier to track and register scans if you circle around or move steadily toward a point of reference (building or another object). More complicated scan orders result in more difficult comprehension during the scan registration process.

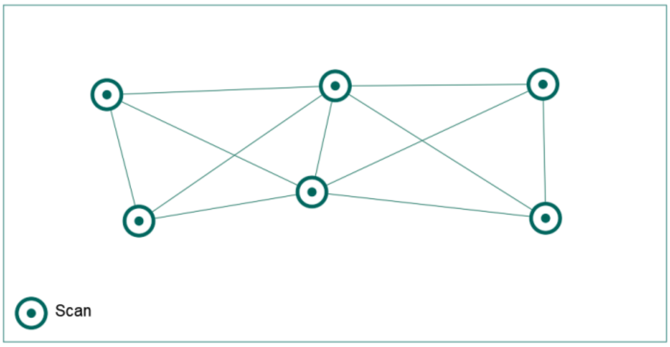


Figure 14-2 Preferable Scan Network

14.3.Data Processing

The raw measurements from a TLS are a point cloud with color and intensity information in each point, at the reference system of the scanner. If wavefront ToF is used then each point is also classified according to its return.

The raw data must be processed before they become useful information. The first step is the registration of all scanning stations in a common reference system. The second optional step is the classification of points into classes. This is an automated step supported by AI techniques, using color and intensity information, simple geometric criteria, or a combination of both. The final step is geometric entity extraction, which is a semi-automated process supported by CAAD software modules. The final product may be a unified point cloud, a CADD or a BIM file.

14.3.1. The Point Cloud File

Most current commercial TLS are using their own proprietary format, and are accompanied by dedicated software for reading scanner data, preprocessing and exporting in more common and interoperable formats. Scanners proprietary formats usually hold additional information and file structure, including photos taken, position and orientation data, multiple returns if available, etc. The common formats hold less information in a more structured manner.

14.3.1.1. For point cloud data

XYZ

This is a non-standardized set of files based on Cartesian coordinates XYZ. Although it is ASCII and there is wide compatibility across programs, one must define columns and units when importing the file.

PTS, PTX

These formats although, are also quite common nowadays and supported by most BIM s/w. They are useful because they're easily converted and manipulated.

E57

This is a vendor-neutral file format for TLS point cloud storage. It can also be used to store images and metadata produced by laser scanners and other 3D imaging systems. It is compact and widely used. It also utilizes binary code in tandem with ASCII, providing much of the accessibility of ASCII and the speed of binary. E57 can represent normal, colors and scalar field intensity. It is widely accepted as standard and most software support import and export in this format. Probably the best option nowadays.

LAS, LAZ

LAS was introduced by the American Society of Photogrammetry and Remote Sensing (ASPRS). It is a binary file format, with fixed records, mainly for aerial LiDAR surveys. It can also contain several metadata for each point, relevant to the nature of aerial LiDAR, such as the direction of flight, return, angle from nadir, etc. LAZ is a compressed version of LAS.

14.3.1.2. For triangular mesh data

OBJ

This file format was developed by Wavefront technologies, and has been adopted by a wide range of 3D graphics applications. It is a simple ASCII data format that represents 3D structured geometry, like points, triangles, normals, color and texture. There are some proprietary binary versions of OBJ, but it is considered one of the most generic formats available.

PLY

This format, known also as Stanford triangle format, was inspired by OBJ and purpose-built to store 3D data. PLY uses lists of nominally flat polygons to represent objects. The goal was to add extensibility capabilities and the ability to store a greater number of physical elements. The result is a file format capable of representing color, transparency, surface normals, texture, coordinates and data confidence values. There are both ASCII and the other binary.

14.3.2. Point Cloud Processing

Most of the scanner vendors provide their own point cloud processing software, at least for preprocessing raw data from the scanner. In addition, several other companies have developed point cloud processing software. By exporting the point clouds in the XYZ file format, point clouds from any scanner can be analyzed with any third-party software.

The basic functionality of such software includes cleaning, filtering, segmentation, classification, sectioning, rendering with texture, vectorization, image-based outputs, animation and visualization.

The following fundamental editing/analysis features are embedded in most of TLS's software packages:

- Point cloud colorization from the laser images.

- General point cloud visualization, including pan, tilt and zoom.
- General point cloud editing, including adding and deleting points, noise removal and point decimation.
- Ability to stitch together multiple scans.
- Ability to handle various import and export formats (to CADD programs, for example).

The following advanced features are found in some, but not all of the software packages, or are part of additional modules:

- Ability to create a triangulated surface (Triangulated Irregular Network, or TIN).
- Ability to best-fit lines, planes and other shapes to point cloud clusters.
- Ability to make profiles and cross sections through a point cloud.
- Ability to make measurements such as distances, angles, areas and volumes.
- Ability to register scans, including the automatic detection of targets.
- Perform solid modeling (volume generation) based on user-defined lines, planes and other surfaces as bounds.
- Perform automatic extraction of standard shapes from cloud.
- Have edge detection technology to determine boundaries of solids, planes and other shapes.
- Ability to drape a digital image over a triangulated surface.
- Automatically compute a full 3D polygonal mesh (not 2.5D) from a point cloud.
- Ability to integrate scans with floor plans, engineering drawings of objects and surveyed information.
- Ability to visualize the space in 360 ° image spheres and VR.

By default, TLS are measuring points in their own internal system, usually with reference point 0,0,0 at the rotation center of the scanner, and a random azimuth. Some modern scanner implementations are equipped with positioning systems varying from GNSS antennas, compass, barometers for height estimation, IMU, or even cameras for vSLAM. Another category of modern scanners provides total station functionality, with the ability to manually aim at a point and measure angle and distance, thus allowing for traverse or resection functions for the estimation of TLS positioning in the reference system in use.

Some systems allow for a draft estimation of position and orientation. If the estimation is accurate enough, then it can be used for direct georeferencing of the point cloud during the data acquisition phase. Otherwise, the position, height and orientation data can be used as initial estimations for further fine registration.

14.3.2.1. Point cloud preparation

Some parts of data recorded in the point cloud can be misleading or inaccurate. In order to avoid influencing the process of alignment between distinct sets of point clouds, steps should be made to remove this information from the final datasets. Erroneous data can be recorded cause of, highly reflective (mirrors, polished metal), absorbent (black) and translucent (clear glass), moist surfaces. These surfaces affect the laser system's capacity to receive a return pulse surface. When facing these problems, as a solution the user could color, powder or tape these surfaces to limit the implications of scanning. Other obstructions may occur because of small or bigger moving objects, humans passing by, tree leaves moving because of windy weather etc. could influence the quality of the scans. The removal of the erroneous data from the recorded data can be performed by the corresponding software, accompanying the TLS device. Additionally, caution should be taken when facing such implications as valid data can be accidentally removed along with the erroneous data (TMR, 2021).

14.3.2.2. Registration using traverse

Some TLS allow to be positioned on known points and set the azimuth towards a known point. Some others support basic total station functionality. In such cases, the registration of the PCs is done on the site directly, by setting the TLS over known points and backsight to a known point. The point cloud is recorded in the reference system of the TLS and needs no further processing after downloading.

14.3.2.3. Registration using targets

The most common registration process is using targets. Spherical targets are the most common and can be recognized by most point cloud processing software. They can be either used as simple tie points among adjustment scans or assigned coordinates to register the point cloud to a known reference system. Coordinates of the targets can be estimated using total stations. Three targets among neighboring point cloud are enough to co-register them together.

Depending on the scanner technology (ToF or Phase shift), the targets may vary among constructors. A plain black and white A4 sheet may be used as target for phase shift scanners, while ToF needs specialized circular targets with reflective surfaces (Figure 14-3). Such targets are usually automatically recognized by the postprocessing s/w of the TLS scanner and can be assigned coordinates as measured by total stations, hence geo-registering the point cloud. These targets can be placed around the scanning area and close enough to the device during the process of scanning by the TLS instrument.

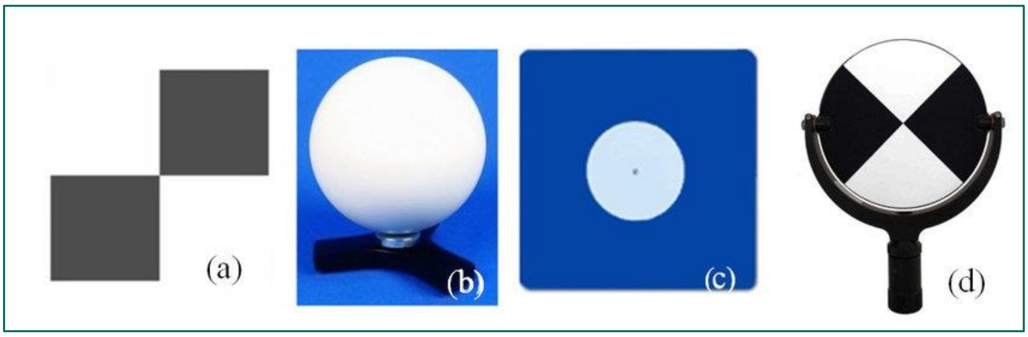


Figure 14-3 Several Types of TLS Targets (Fryskowska-Skibniewska, 2019)

14.3.2.4. Computer assisted registration

The main algorithm used by all software for point-cloud to point-cloud registration is Iterative Closest Point (ICP) (Zhang, 1994). It works directly on point clouds, without the need of them being transformed into surfaces beforehand. The algorithm iteratively revises the initial draft transformation (combination of translation and rotation) needed to minimize several distances from the source to the reference point cloud, i.e., the sum of squared differences between the coordinates of the matched pairs. Nevertheless, it may fail, if there is not enough overlap (> 60 %) among neighboring point clouds.

Provided the TLS has some position/orientation sensors on board, and each scan has metadata about their position, the ICP process can start without human intervention, using ICP to refine the point cloud registration. If the on-board sensors fail to provide a good initial approximation, then this can be done manually and the process restarted.

14.3.2.5. Adjustment

An inner-constrained least squares adjustment, with no control points, allowing the measured scan network to reveal its internal reliability and accuracy shall be run on multiple scans stitched together when using computer assisted registration with cloud – cloud registration.

To confirm the quality of scan-to-scan adjustment, the residuals should be reviewed to ensure that there are no network links with residuals that have to be handled or removed. A check for possible outliers among the link residuals has to be performed. Any outliers should be removed, and adjustment has to re-run until all links are within the defined tolerances of accuracy and reliability.

After the minimally constrained adjustment has been completed, a fully constrained least-squares adjustment shall run, including positions of all known control points used for the registration process, along with all the respective scan links. This adjustment is subject to the same steps as above. Possible outliers shall be checked among the control points and the scan network links. Outliers have to be removed and the constrained adjustment has to re-run until all control points are within the defined tolerances of accuracy and reliability. Final link residuals have to be checked of being within the accepted range. In the other case, the survey is rejected and the total survey shall be repeated.

14.3.3. Interoperability with CADD Software

The time and cost to process the unstructured point cloud to become a complete CADD model is often overestimated. Nowadays interoperability has greatly improved allowing traditional CADD software to handle millions of points, which was not a given a few years ago. Nevertheless, transforming an unstructured point cloud in a 3D CADD model is still a semi-automated process, which requires extensive manual intervention.

CADD software principally includes Bentley and Autodesk suites which provide dedicated modules for point cloud interoperability. Other software suites are also available, either as an add-on to the aforementioned CADD or as standalone solutions. All major TLS vendors such as Leica, Trimble, Faro, etc., provide software with point cloud to CADD functionalities.

Such software provides the functionality to geometrically segment point cloud and extract basic geometric primitives, such as cones, planes, cylinders, etc., using semi-automated processes for recognizing and fitting more complicated shapes such as pipelines, pipeline joints, faucets, etc., selected from extended dedicated 3D object libraries.

In terms of modeling man-made objects (buildings, bridges, roads, tunnels, harbors, etc.) to create a 3D city model of buildup environment, the Level of Detail concept has been introduced by Karlsruhe Institute of Technology (Figure 14-4).

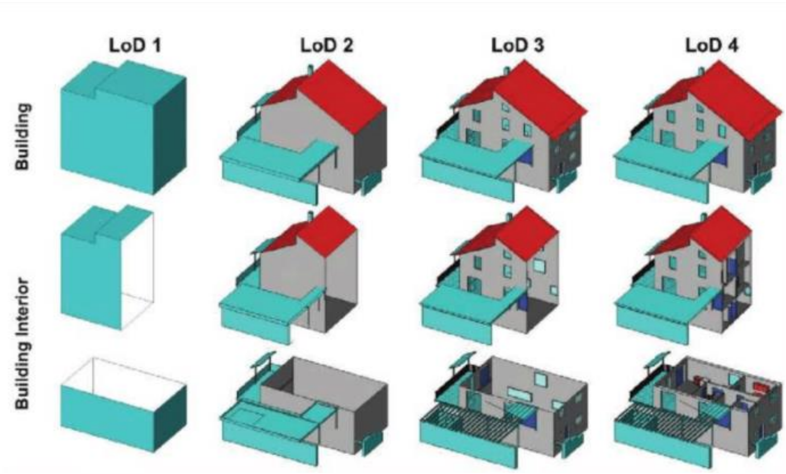


Figure 14-4 Different Levels of Detail (LoD) for Buildings According to CityGML. LoD 0 is not Represented Since it is not a 3D Model but Simply the Building Outline with Height as Attribute (OGC, 2012)

14.4. Accuracy and Precision

Overall accuracy of a complete LiDAR survey depends on several aspects such as:

- TLS specifications and type.
- Accuracy and resolution (density of points).
- SNR.
- Range, inclination and material of the object.
- Site and weather conditions.
- Registration accuracy.
- Environmental conditions such as heavy rain, snow, and fog will significantly deteriorate their accuracy.

Therefore, it is not easy to predict overall accuracy, since it is correlated to the settings and the acquisition strategy. It is therefore advised to specify correctly the survey during commissioning. Checking the deliverables is challenging, especially in terms of completeness. Accuracy is easier to check using industrial grade total stations, to verify distances among points in the delivered 3D model.

Typical density and precision with respect to scale are given in Table 14-1.

The precision is dictated by scanner type and specs. Nevertheless, the precision of each point can be regulated from scanner settings, since most modern scanners may offer multiple passes over the same point to reduce SNR, at the cost of slower scanning speed.

Typically, mechanical LiDAR achieves a range accuracy at centimeter level (1 - 8 cm) for the full scale of their range. Lower-grade LiDAR sensors, with significantly lower price, achieve a range accuracy at the centimeter level (usually 2 - 20 cm), for their full range, which is usually less than the mechanical LiDARs. Still, such accuracy is appropriate for obstacle avoidance and object detection applications.

Table 14-1 LiDAR Expected Precision and Density with Respect to Scale

	Scale	Point Density (mm)	Precision of Measurement (mm)
Close range	1:5	0.5	0.5
	1:10	1.0	1.0
Terrestrial	1:20	2.5	2.5
	1:50	5.0	5.0
	1:100	15.0	15.0
Aerial	1:200	30.0	30.0
	1:500	75.0	75.0

14.5. Delivery

Documentation and final deliverables of surveys are an essential part of the project. The documentation of a laser scanning project must show a clear data lineage from the published primary control to the final deliverables. Different projects require different types of deliverables, which can range from a standard interpreted CADD product to a physical 3D printed scale model of the actual object.

Considerable office time is required to extract solid entities from a point cloud to a CADD or BIM usable format. The ratio of field time to office time will vary greatly with the complexity of the scanned roadway and features. Resources for data extraction (computers, software and trained personnel) must be available. Deliverables specific to TLS or any other LS surveys may include, but are not limited to:

- Raw PCs in a specified format, with intensity and color information for each point.
- Cleaned PCs.
- Registered and merged point clouds.
- Decimated point clouds to much requested density.
- Classification of points.
- Triangulated mesh model with texture.
- Roadway Design files.
- CADD files.
- Digital photo mosaic files.
- 3D printing technology physical scale models of the subject.
- Survey report and QC/QA files.
- Geospatial metadata files.

15. Mobile Mapping

Mobile Mapping Systems (MMS) are devices that collect geospatial data from a mobile platform, typically fitted with a range of cameras, LiDAR, Radar, echo-sounders, or any number of remote sensing systems, for land, air or marine applications. The uptake of MMS is fueled by the demand for fast and cost-effective data acquisition for large areas, along with technological developments which address this demand. Two developments are especially important in this context: LiDAR sensors and precise navigation. The common feature of MMS is that they combine all sensors necessary to solve a specific problem, mounted on a common platform. By precise synchronization of all data streams, data collection for a specific application is possible, even in real time.

The mobile platform may vary from aircraft, car, surface or subsea vessel, manned or unmanned. Therefore, all types of UAV, drones, Unmanned Surface Vehicles (USV), Remotely Operated underwater Vehicle (ROV), cars, robots, even humans can be considered as moving platforms. By definition airborne photographic mapping using an on board GNSS and IMU system for direct georeferencing is a MMS, although the use of additional data from GNSS and IMU are optional for the photogrammetric process. Airborne LiDAR is also an MMS, but in this case, GNSS and IMU data are integral to the process of georegistration of LiDAR data, which would be useless otherwise.

The primary output from such systems includes 3D point clouds, georeferenced images and video, GIS data and digital maps. Hence, MMS are an important source for various applications, including, but not limited to: indoor and outdoor 3D modeling, capturing of GIS data, disaster response high-detail 3D maps, and autonomous vehicles (Elhashash, et al., 2022).

MMS are being used in several land, marine, aerial, and indoor applications:

- Asset and infrastructure management/maintenance.
- 3D imagery for online mapping tools, street level views and mobile apps using map data.
- Road mapping and highway facilities.
- Assessment and monitoring of road surface.
- Rail and utility corridors.
- Pipeline surveys (Bridge, tunnel, etc.).
- Indoor mapping for BIM and Cultural Heritage monuments/sites.
- Aerial mapping surveys with manned or unmanned aircrafts, using either camera and/or LiDAR.
- Mining (digital terrain models, deformation monitoring, topographic maps, exploration, mine design, volume measurements etc.).
- Urban planning and real estate.
- Smart cities, digital twin cities.
- Agriculture – crop management.
- Hydrographic surveys – seafloor mapping, underwater detection/exploration.
- Creating virtual environments for games, movies, VR simulations.

Given the variety of applications, there is a wide variety of implementations that fall under the generalized MMS category. An extended, but not exhausted, current list of MMS is included in (Elhashash, et al., 2022). Selection of the appropriate MMS is depended on the application and its unique characteristics.

15.1. Equipment

MMS are typically comprised of two main types of sensors; sensors for positioning and sensors for data collection. Given sensors' availability and variety (Table 15-1), there are several MMS implementations and categories.

All sensors mounted on the mobile platform must be time synchronized (Figure 2-7). Because some of the sensors are recording data in different frequencies, they might not necessarily be synchronized in the sense that all of them acquire data at the exact same time, but they are all referenced at the same time frame, so they can be associated during post-processing. The synchronization accuracy needed is dependent on the required system performance and on the maximum allowed speed with which the survey vehicle moves.

Table 15-1 Most Common Types of Sensors on Board MMS

Sensor Name	Sensor Data	Characteristics
Large format, aerial cameras	Geometry / Description	Excellent geometric accuracy.
Multi spectral cameras	Description	Adds rich descriptive information for classification.
Video cameras	Positioning / Description	Stereoscopic or monocular, used for Simultaneous Localization and Mapping (SLAM) and visual odometry. Data from cameras may also be used for feature description.
Laser profilers, Laser scanners, LiDAR	Geometry / Description	Measures sensor to object distance. After combination with positioning information, the final product is a unified point cloud. After combination with cameras, a colored point cloud can be exported.
Ultra-sonic sensors	Geometry / Description	Used either for object-sensor distance measurement or cross-section profile measurements, mainly for marine applications.
GNSS	Positioning	High accuracy positioning in real time (navigation) or in post-processing operation.
IMU	Positioning	Position and orientation information. Low accuracy in position, low to high accuracy in orientation, depending on the accuracy grade of the system.

Sensor Name	Sensor Data	Characteristics
DMI, odometers	Positioning	Additional real time information about distance traveled.
MBES	Geometry	Echo sounders for bathymetry. They collect points on the sea bottom.
USBL, SBL, LBL	Positioning	Acoustic positioning systems providing position information, in replacement of GNSS for underwater surveys.

Typically, commercial MMSs can be classified (based on their hosting platforms) into handheld, backpack, trolley, and vehicle-based. Some platforms are designed to work indoors without relying on GNSS, while others can work indoors and outdoors. Platform poses limits in terms of space and weight thresholds for data collection sensors. Platform movement characteristics, may also necessitate the use of specific positioning sensors, to better model/predict actual routes and orientations during acquisition mission. The use of the MMS in GNSS signal deprived environment (indoor, underwater, etc.), is critical since another positioning system must replace its functionality.

In the following subsections, an overview of positioning and data collection sensors is provided, as well as the respective sensor fusion approaches.

15.1.1. Cameras and Vision

The variety and affordability of modern cameras, readdresses design concept of MMS through redundancy and opens the way towards new and flexible implementations of MMS. Combined with algorithms and libraries from Computer Vision and Robotics it can provide flexible post-processing tools, increasing their potential and supported by readily available software tools.

In the form of multi-spectral cameras, they provide additional layers of information, not available from optical cameras. Such information is used for feature classification in agriculture and forestry applications, to detect illnesses in trees/crops or ripe stage for cropping.

The usage of cameras in MMS is quadruple:

- Texturing collected 3D data.
- 3D measurements or 3D dense point reconstruction and meshing.
- Sensing and classification of features.
- Providing positioning.

15.1.1.1. 360° and fisheye cameras

The 3D measurements of an MMS rely on LiDAR sensors, while the images are primarily used to provide colorimetric/spectral information. Depth estimation and 3D data collection of MMS mostly rely on LiDAR data collectors because of their low noise characteristics and excellent description of 3D shapes, especially in edges. Therefore, although excellent devices are used for navigation or obstacle avoidance, for example in automotive industry, they lack visual information which is important for human interpretation. Cameras are being used in tandem with LiDAR sensors to colorize the point cloud they provide, or texture the 3D mesh (surface) created from LiDAR point cloud data.

Developments in camera technologies has led to 360 ° cameras, which are enclosures or several calibrated cameras under a single body. Such cameras can record in high rate/detail video or stills. The final output may be just 360 ° video/stills content suitable for VR applications such as Google streets, but it is also perfect for capturing color information for mobile mapping applications. Hence, such cameras (Figure 15-1) are typical payload of MMS, suitable for colorization of the point clouds.

Much cheaper fisheye cameras can be used instead. These are single lens cameras, covering a very wide field of view (180 ° or more). Usually a combination of two, may provide a full 360 ° panorama, but of much lower quality than the aforementioned dedicated 360 ° cameras.

Mosaic 51

Source: www.mosaic51.com

Ladybug5+

Source: www.flir.com

Figure 15-1 Examples of 360 ° Cameras for Mobile Mapping

15.1.1.2. 3D data collection

Although 3D data collection in MMS, mostly relies on LiDAR the advent of computer vision and robotics, provide a viable alternative with cameras. In fact, several applications rely only on stereo cameras to recover 3D information, instead of more expensive LiDAR sensors. Such cameras are machine vision cameras, with known pre-calibrated relative orientation, synchronized with high accuracy, with overlapping areas. These cameras can provide depth and scale with better accuracy, so that to be fused with LiDAR data. Nevertheless, their depth accuracy depends on their geometric characteristics, resolution and software algorithms.

In applications, a limited number of points/features must be recorded, or only a few distance measurements are necessary, cameras are a more efficient tool. Such applications might include infrastructure recording (electricity poles, light poles, traffic lights, etc.), real estate (façade length, building height, etc.), road maintenance, VR applications and similar.

In cases where the 3D environment should be captured, SfM, MVS and Simultaneous Localization and Mapping (SLAM) algorithms, can provide 3D textured meshes, although at the cost of extended post-processing times and significant noise.

On the other hand, they are excellent if a few selected points are to be measured manually with stereoscopic observation by human operators. They can still provide texture information for 3D data fusion with LiDAR sensors.

15.1.1.3. Feature classification

The advent and wide adoption of AI into autonomous driving, has affected geomatics as well. Redundancy of data from several data acquisition sensors, allow for extended research into the automatic classification of features in MMS. At the same time the huge data gathered by such systems provide a challenge for human interpretation, mainly in terms of time. Therefore, tools that could classify data, and reduce or even eliminate the human burden, have been introduced.

Apart from LiDAR intensity, RGB or panchromatic cameras, multispectral cameras acquire additional information towards that end. Most common applications are in farming and forestry. Former applications include crop disease detection, crop stress detection, ripe index evaluation in vineyards, all used for precision agriculture, i.e., targeted interventions. In forestry, AI and classification are being used to automatically count the location of trees, along with fundamental measurements (height, canopy width), useful for forest monitoring, inspection and tree disease detection.

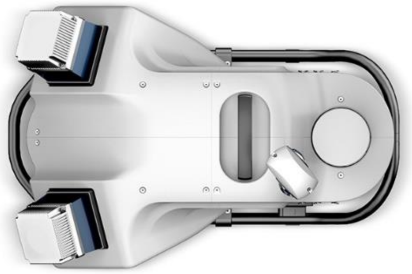
It should be noted that AI research is focusing on fundamental classification of LiDAR points into ground/building/vegetation, to prepare data from LOD2 city model (or digital twin city) generation. Research is also engaged in recognition of façade openings (doors, windows, balconies), to prepare data classification towards LOD3 city models.

15.1.1.4. Positioning

Stereocameras (Section 15.1.1.2), or single cameras, can support stereo or monocular positioning, through SLAM. SLAM was developed under the robotics field, developed in the 1980s, which enabled robots to navigate autonomously through unknown environments, using several technologies and sensors. Nowadays, sensors used in SLAM are mainly LiDAR profilers and cameras.

Cameras in particular rely on correlation of each frame, to the previous one, hence registering each new frame to the previous, in terms of location and orientation (5 or 6 parameters depending whether on monocular or stereo cameras are used). When SLAM is only performed with cameras, then the route is not georegistered and the positioning is only relative. Hence, they cannot provide a stand-alone system for positioning because the registration is relative and the route suffers from drift, which is the accumulated error over each frame.

Camera based SLAM systems can provide redundant information for fusion with other positioning systems, or provide standalone positioning and orientation, relative to the last position with recorded GNSS signal, when signal is shortly lost, in indoor or subaquatic applications.



Leica Pegasus TRK700 neo

Source: https://leica-geosystems.com/

Trimble MX50

Source: https://geospatial.trimble.com/

Figure 15-2 Examples of Car Mounted MMS

15.1.2. LiDAR

LiDAR, or light detection and ranging, is an optical instrument that uses directional laser beams to measure the distances from objects. It provides individual and accurate point measurement on a 3D object; thus, when combining many of these measurements the shape and surface characteristics of objects can be recovered. Fundamentals of LiDAR technology and static applications have been covered in Chapter 14 of this Volume.

As a 3D measuring tool, it has many desirable features, such as high accuracy, high acquisition rate, dense 3D recorded information, exhibits invariance to illumination, and can partially penetrate sparse objects like canopies. When used as an MMS subsystem, it requires a suite of highly accurate and well-calibrated navigation system to reliably retrieve 3D points, the installation and cost of which, in addition to the already expensive LiDAR sensor, make it a high-cost means of collection. Since the MMS is moving, there is no need for 360 ° LiDAR scanners (like those described in Chapter 14 about Laser Scanning), but only LiDAR profilers (or rotating). Modern MMS systems incorporate at least one or more LiDAR profilers, as the main 3D data acquisition sensors. By moving these profilers along with the mobile mapping system, it is possible to record dense point clouds, along with intensity of reflected signal, or even multiple returned signals. Any subsequent colorization relies on camera information.

Survey grade LiDARs which are used in MMS, may achieve accuracy to millimeter level, when stable. When moving on the mobile platform, their relative accuracy remains the same, but the final absolute accuracy depends on the positioning sensors' accuracy.

The rate of data acquisition in LiDAR profilers (Figure 15-3, Figure 15-4) is related to vehicle speed. If it's high, then there is less time to acquire data, hence more LiDAR beams inside the profiler are needed to ensure that the object of interest is properly described by sufficient points. For instance, a 32-beam LiDAR could be sufficient for a vehicle moving at a speed of 50 - 60 km/h, but a LiDAR with 128 beams is recommended for higher speeds, up to 100 - 110 km/h, so that the acquired data have adequate resolution. Other operational criteria of LiDAR can be also important and should be considered depending on an application (e.g., long-range LiDAR may be unnecessary for indoor applications.). Weight and power consumption are also of interest when the LiDAR is intended for backpack use.

Hand held Zeb Horizon

Source: https://geoslam.com/solutions/zeb-horizon/

Backpack ViAmetris BMS3D-HD

Source: https://viametris.com/backpack-mobile-scanners/

Figure 15-3 Examples of Small and Lightweight Hand Held and Backpack Forms of MMS

The range of some LiDAR operational parameters can be seen below:

- Range accuracy 0.1 to 8 cm.
- Range from 1.5 to 800 m.
- Number of beams 16 to 128.
- Points per second 300,000 to 3,500,000 points per second, related to number of beams

Use of solid-state LiDAR in an MMS is a promising direction, because of its lower cost than rotating LiDAR, and there might be commercial implementations in the near future.

15.1.3. Position and Navigation

Although a number of different systems are used in general navigation, the rather stringent requirements in terms of accuracy and environment make the integration of an INS with GNSS receivers the core of any sensor combination for an accurate mobile mapping system for short range applications. This combination also offers considerable redundancy and makes the use of additional sensors for reliability purposes usually unnecessary. Car mounted MMS usually incorporate a DMI, as additional information for distance.

All information from position and navigation sensors is fused using a variation of the Kalman filter, optimized for the kinematic characteristics of the mobile platform.

15.1.3.1. Global navigation satellite systems

The signals from satellite constellations such as GPS, GLONASS, Galileo and BeiDou, are utilized by the GNSS receiver to compute the position, velocity, and elevation of the antenna center. More about the fundamentals of GNSS have been covered in Section 2.9.3.

The main benefits of GNSS system are:

- Data collected have global reference, of utmost importance for mobile mapping applications.
- Because of the external dependence of the system, it is unaffected or less prone to accumulated errors.

On the other hand, there are some limitations:

- Signal is limited or nonexistent in complex urban environment next to high buildings, tunnels, underwater, indoor environment, or under trees.
- RTK-GNSS may be able to provide real time positioning, but to attain its full accuracy potential, post processing is required to minimize errors from receiver's noise, pseudo-range, carrier phase, doppler shifts, atmospheric delays, etc.

Information provided by a single GNSS antenna is limited to X,Y,Z and the system cannot provide accurate rotation and orientation data, which are critical for LiDAR profilers and cameras, so that to be georegistered. Also, the theoretical accuracy measures are conditioned to open areas. As such, when collecting 3D data in dense urban areas with narrow streets and buildings or indoor environments, the GNSS signal is heavily impacted by occlusions and signal reflections, so the resulting measurements can be inaccurate even if a fixed solution can be reached. Hence, it requires complimentary sensors when operating under such conditions. In general, the positioning platform of a high-grade MMS is expected to achieve an accuracy of 5 - 50 mm at speeds that can reach the maximum speed of highways (120 - 130 km/h) when considering the integration of multiple sensors. Therefore, it is used as complimentary positioning system used in tandem with IMUs.

15.1.3.2. Inertial measurement unit

IMU, is an independent sensor that records the relative position of the orientation and directional acceleration of the platform. Unlike GNSS, it's a standalone system that doesn't require links to external signal sources, hence independent from external information/sensors.

An IMU consists of an accelerometer and a gyroscope, which it uses to sense acceleration and angular velocity. These raw measurements are processed by an onboard computing unit to provide dead reckoning real-time positioning. Thus, the IMU and computing unit, together with the algorithm as a whole, are also called an Inertial Navigation System (INS).

Main benefits include:

- Capable of navigating in all environments, such as indoors, outdoors, underwater, tunnels, caves, etc.
- A necessary supplemental data source for urban environments where GNSS is unstable.

Limitations are:

- Strong drift from accumulated errors over time, can be dealt with calibration and supplement system for timely correction to true position.
- Limited to short-range navigation.

The grade/quality of IMU sensors is related to the type of gyroscope in use: a majority of light-weight, consumer-grade IMUs use microelectromechanical systems (MEMS), which are affordable but suffer from poor precision and large drift errors (often 10 - 100 °/h). The main categories of gyroscopes are:

- Strategic gyros (0.0005 - 0.0010 °/h or degree per month).
- Navigation-grade gyros (0.002 - 0.01 °/h or degree per week).
- Tactical gyros (1 - 10 °/h or degree per hour).
- Low-accuracy gyros (100 - 10,000 °/h or degree per second).

As mentioned above, IMUs positional information is calculated through dead reckoning approaches. Like many other independent navigation methods, it suffers from accumulation errors, often leading to significant drifts to its true positions. Hence, its measurements will only be accurate for a relatively short period in reference to the starting point. It is able to record relative positions with respect to its starting point, which can usually be dynamically provided by GNSS in open fields. This, as a standard approach, provides more accurate positional information in complex environments mixed with both open and occluded surroundings.

PwrPak7D-E2

Source: https://novatel.com/products

Survey+ v3

Source: https://www.oxts.com/

POS Track

Source: https://www.applanix.com/

Phins Surface

Source: Phins Surface - iXblue

Figure 15-4 Examples of High Grade IMU for Mobile Mapping

15.1.3.3. Distance measurement instrument

DMI generally refers to instruments that measure the traveled distance of the platform. In many cases, DMI is alternatively referred to as odometer or wheel sensor for MMS based on cars or bikes. It computes the distance based on the number of cycles the wheel rotates. Since DMI only measures distance, it is often used as supplementary information to GNSS/IMU as an effective means to reduce the accumulated errors and constrain the drift from IMU in GNSS-denied environments such as tunnels. It requires calibration before use and measures distance, velocity, and acceleration.

15.2. Procedure

Data acquisition is straight forward and vastly dependent on the hardware. Human intervention is only needed to guide/maneuver the vehicle, if this is not autonomous and monitor the data collection. MMS are usually accompanied by route optimization software to ensure full and efficient coverage of the area of interest.

Calibration of systems and synchronization are critical factors in the data acquisition process, along with maximum acquisition speed, in order to avoid limited coverage of data. Exact knowledge of the relative position (level arms) between all subsystems is required for the successful fusion of all data. Sensor calibration and fusion are often performed throughout the data collection cycle. The goal of this is to calibrate the relative positions between multiple sensors, including between cameras, between camera and LiDAR, or among LiDAR, camera, and navigation sensors.

For MMS positioning, the GNSS, IMU, and DMI continuously measure the position and motion of the platform. In most outdoor applications, the main navigation and positioning data are provided by the GNSS satellite to the receiver, and the IMU and DMI supplement measurements where GNSS signals are insufficient or lost. In some specific cases where GNSS is completely inaccessible, such as cave mapping, or subaquatic mapping, the GNSS must be replaced with some equivalent system (Chapter 16).

Data are also being acquired simultaneously by the integrated LiDAR, visual or acoustic sensors to produce accurate 3D point clouds colorized by the images from the associated camera.

15.2.1. Equipment Calibration

Depending on the customization and requirements of the manufacturer, the MMS can be set up in a variety of ways. Nonetheless, the main components, no matter what the configuration of a MMS, should be calibrated according to manufacturer’s instructions. Furthermore, the general guidelines should be followed as described in Section 4.2.

15.2.2. Equipment Care

Unmounted MMS are easier to handle and transfer, since most of the times they are being handled by one person, who can easily move around the area of interest. On the other hand, mounted MMS are heavier, harder to install as opposed to unmounted MMS that are significantly lighter and offer more mobility. MMS devices differ from their installation setups to their mechanisms and technologies adopted. Hence, it is difficult and unwise to mention detailed instructions about such systems and their equipment; nonetheless, no matter the type of MMS devices used, there are some general safety checks and preparations that need to be taken into account.

- A safety check should be completed before and after each mission.
- Abrupt mechanical shock should be avoided. Shocking could result in permanently damaging the equipment.
- All broken or damaged parts should be removed and replaced as soon as possible.
- A check that ensures, all of the screws are tight (referring mainly to heavy, mounted MMS devices).
- All cable connections should be checked as they need to be well connected and damage free.
- The optical surfaces (such as scanning mirror, etc.) of the MMS instruments should be kept clean. Rubbing off dust or debris of the optical surfaces should be avoided as this will scratch the glass and may damage the instrument, instead, the use of compressed gas duster is advised. Before cleaning, the MMS device has to be switched off as well as removing the battery from the equipment. Any dirty or moisture should be cleaned or dried before powering on the MMS equipment.
- Starting a mission before solving any issue that may occur previously with the system, could cause permanent damage to the system.
- All the installation and mount of possible roof rack should be installed according to the manufacturer's instructions.
- This type of equipment is not easy to use, the operator should be trained, experienced and take care to fulfill all requirements and certifications.
- Packing correctly the equipment in the supplied transportation cases, ensures the safety of the equipment during transportation and storage.

15.2.3. Data Acquisition

15.2.3.1. Multiple pass requirements

When MMS are implemented, it is important to apply redundancies in the survey, as they are necessary to produce reliable statistical results. Successful acquisition of complete point clouds requires the pavement to be run in a minimum of three passes. Additionally, multiple passes over a period of time are advised to minimize the errors caused by GNSS shift due to multipathing and shadowing affects. No matter the type or the number of scanners used the practice of at least three independent passes should be applied unless specific circumstances dictate otherwise. Each lane must be scanned a minimum of three times if the scan of one lane does not completely overlap the scan of the adjacent lane. When facing multi – lanes it is advised to consider the most appropriate for scanning (TMR, 2021).

15.2.3.2. Carriageway pass requirements

MMS require a plethora of data and information in order to be accurate. This principle should be met no matter the type and combination of carriageways and lanes. These types and combinations along their preferred practice may be (TMR, 2021):

- Single carriageway and multiple lanes (Figure 15-5): There should be one pass in each direction of travel. To provide enough coverage, density, and redundancy in the data collected, a third pass is needed in either direction.

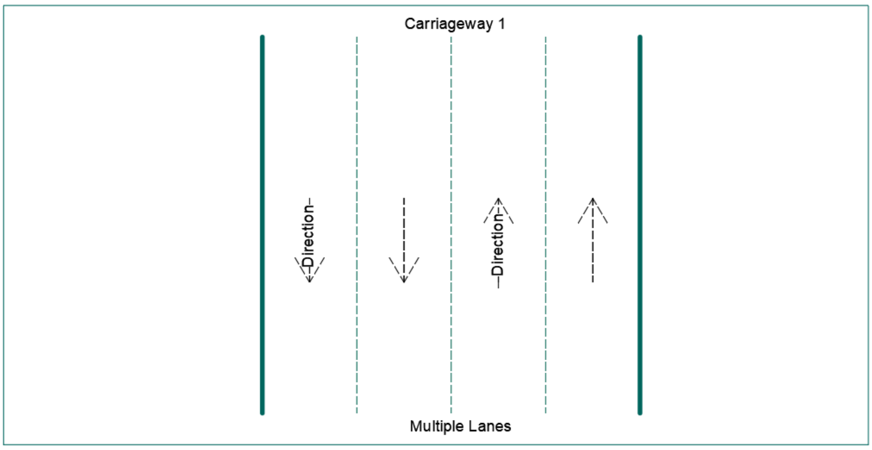


Figure 15-5 Single Carriageway, Multiple Lanes

- Single carriageway and divided lanes (Figure 15-6): A minimum of three scans per direction should be applied when no sufficient overlap exists between opposite direction scans due to the barrier (TMR, 2021).

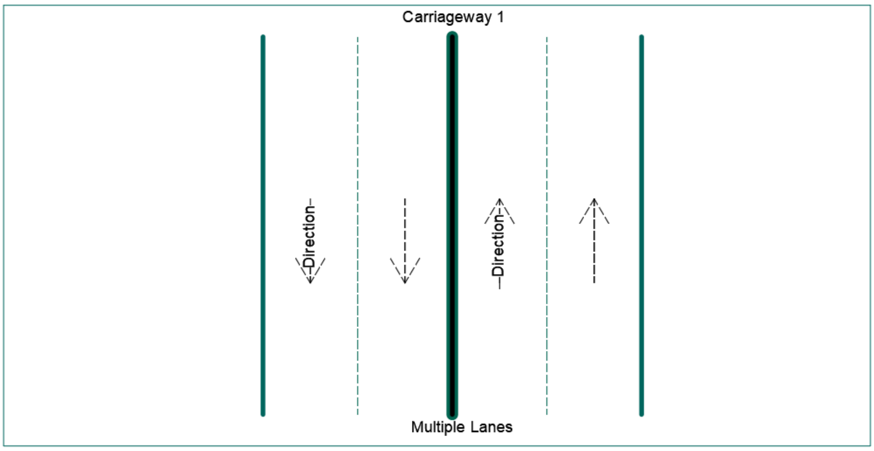


Figure 15-6 Single Carriageway, Multiple Divided Lanes

- Dual carriageway and multiple lanes (Figure 15-7): On dual carriageway multiple lanes roads where the scans from each carriageway do not fully overlap, each carriageway shall be scanned a minimum of three times (TMR, 2021).

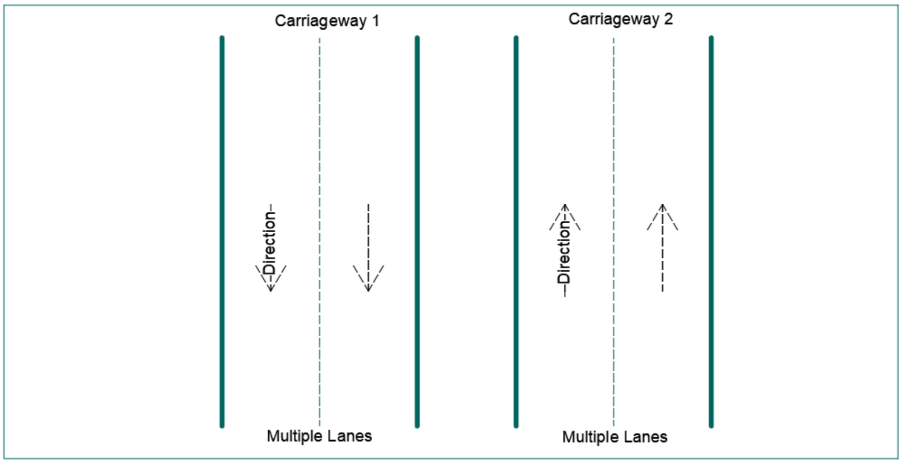


Figure 15-7 Dual Carriageway, Multiple Lanes

15.2.3.3. Poor GNSS environments

When poor GNSS signals are obstructing the completion of the mission other strategies should be applied. IMU and DMI technologies can be implemented as long as their accuracy meets the minimum required accuracy of the project.

15.2.3.4. Data overlap

Steps and gaps in the given point cloud, whether horizontal or vertical, are unacceptable. Such phenomena may occur on independent passes on road intersections or where runs have been stopped and begun on the road. To guarantee that a consistent and thorough point cloud is supplied at these sites, control points should be employed. When the current project should be matched to an older MMS project, the project proposal will include all pertinent information and data needed to guarantee a seamless match of the two datasets.

15.2.3.5. MMS factors

In general, the quality of the supplied point cloud is influenced by the environment, coloration, shadowing, density, and pattern of the point cloud. It is getting increasingly harder to create specifications that cover every component of every system because there are so many different MMS hardware vendors and manufacturers, while customized hardware configuration can be also applied. The aim is to provide enough details about setting up the MMS instruments as successfully as possible. To achieve the requisite accuracy in feature recognition, the provided point cloud should be of adequate quality. Below, information about environmental factors, coloring, density, accuracy and range is provided (TMR, 2021).

- Environmental Factors: Different hardware will react differently to various environmental factors. For instance, moist surfaces affect a laser system's capacity to receive a return pulse. The quality of the point cloud captured will vary according to the weather, including rain, standing water, and even a thick dew. All conditions should be taken into consideration in order for the best possible technique to be applied.

- Point cloud colorization: two main techniques are used. The first technique colors the point cloud with Red Green Blue (RGB) values that have been assigned to individual points within the point cloud from imagery. The second technique uses an intensity value derived from the strength of the return signal to the laser scanner. This provides a greyscale colorization derived from the intensity.
- Density and scan accuracy: While some systems can capture over a million points per second, others can only do a small portion of this. Selecting the appropriate point cloud accuracy depends on the required accuracy of the project. Furthermore, another factor that needs to be considered is the vehicle speed which significantly influences the final point density and accuracy of the point cloud.
- Range: Scanners and sensors, vary in accuracy and capture ranges, all the way up to 200 meters, on more advanced equipment. Additionally, the quality of IMU, DMI and GNSS systems can influence on how far from a scanner a point cloud corresponds to the minimum required project’s accuracy. In most cases, the portion of the point cloud that satisfies the project's accuracy requirements will often be much closer than the maximum range the MMS system can capture.

15.2.3.6. Imagery capture

When MMS supports the technology of capturing geo referenced images, they should be used. When capturing images there are some requirements that need to be met (TMR, 2021):

- Images captured should describe all the area of the project.
- Image capture should occur simultaneously with the point cloud capture.
- The images should cover the entire area of the project.
- Images taken in the early morning and late afternoon may suffer from under and over exposure which is likely to cause problems. Usually, the most optimal period of the day to capture images is between 8 am to 4 pm.

15.3. Data Processing

Mapping approach using MMS comprises from the following steps:

1. Calibration.
2. Synchronization.
3. Data acquisition.
4. Kinematic Modelling.
5. Georeferencing.
6. Integration and data fusion.
7. Quality control.
8. Data flow optimization and automation.

Additional to the real time sensor calibration during acquisition, sensor output must be fused as a postprocessing step to achieve more accurate positional measurements. These serve multiple purposes: more accurate localization, more accurate geometric reconstruction, and data alignment for fusion. It is being performed by software and includes kinematic modelling forward and backward, detection and verification of closed loops to ensure seamless 3D data alignment.

The integration of GNSS, IMU, and DMI for precise localizations, involves the fusion of all sensor information to output the estimated positions through optimal statistical/stochastic estimators. A typical algorithm used for this purpose is the Kalman filter, which uses continuous measurements over time with their uncertainties, along with a stochastic model for each sensor and the vehicle dynamic characteristics, to estimate the unknown variables in a recursive scheme. Kalman filter is the simplest dynamic estimator that assumes linear models and Gaussian random noise of observations. As such, it is often readopted through Extended Kalman Filter for nonlinear models.

The final step in post processing is the use of the estimated route to reference LiDAR scans with images and all related data in a local coordinate system. Georeferencing them refers to determining their geodetic coordinates, mostly based on fused GNSS/IMU/DMI positioning data. Georeferencing includes the estimation, if not known from precalibration, of the orientation (boresight) and position (lever-arm) offsets with respect to GNSS and IMU. The boresight and level-arm parameters define the geometrical relationship between positioning and data collection sensors. There are two approaches to perform georeferencing: (1) the direct approach, which uses only GNSS/IMU data, or (2) the indirect approach, which uses GNSS/IMU data in addition to GCP and BA for refinement. The direct approaches are less demanding, since they do not require GCP, and they can achieve accuracy in the decimeter to centimeter levels. Indirect approaches can provide more accurate (centimeter-level) and precise results where typical surveying methods like GCPs and BA are adopted. However, they are very expensive, and their accuracy may vary based on the GCP setup (i.e., position and number of GCPs).

15.3.1. Point Cloud Cleansing

The basic principle of the MMS is to capture as much as possible information around a specific area of interest. Scanners and other sensors depict the existing environment at the exact time the capture process took place. Often information captured in the point cloud is invalid and inaccurate. Actions should be taken to remove this information from the final datasets, otherwise process of alignment between different sets of point clouds may be influenced. Usually, the redundant and invalid information of points in the point cloud refers to moving road vehicles passing by at the time of the capture process, other small or bigger moving objects because of unfavorable weather conditions such as tree leaves moving because of windy weather etc. Care must be taken to prevent deleting any valid characteristics from the data that appear to be incorrect. It is advised that instead of deleting data, it be moved to other files or labeled in a way that makes it possible to distinguish it from the clean data (TMR, 2021).

15.3.2. Decimating Point cloud

Point cloud decimating is used to match the requested density. A decimating process should be applied in order to reduce the created density to more acceptable proportions when many point clouds are combined. Normally, the density of the joined point cloud should be higher than that of a single point cloud when a decimating technique is employed, but care should be given to ensure that the ground points remain in the final delivered product (TMR, 2021).

15.4. Accuracy and Precision

The expected accuracy of MMS designed for land applications, varies depending on the platform they are mounted to. The mounting platform also reflects the application that they are designed for, such as indoor, outdoor, BIM, city modeling, facility management, etc.

Accuracy and precision of such systems can be found in Table 15-2.

Table 15-2 Indicative Characteristics of Land MMS

	Type	Camera	Max Range	IMU/GNSS	Accuracy
Car mounted	Outdoor	360º or similar	80 - 650 m	√/√	± 2 - ± 5 cm
Handheld	Indoor & Outdoor	Optional	100 m	×/×	± 0.2 - ± 8 cm
Wearable	Indoor & Outdoor	360º or similar	100 m	√/ optional	± 1 - ± 5 cm
Trolley	Indoor	360º or similar	100 m	√/ optional	± 0.15 - ±1.5 cm

15.5. Delivery

Deliverables of maritime MMS are described in Section 16.8, deliverables of aerial LiDAR surveys are described in Chapter 6 and Section 13.4. Deliverables of land MMS are similar to those describe in Section 14.5 of this Volume.

Given the amount of data acquired in such surveys, as well as the complexity of data structures, usually vendors provide holistic solutions for post-processing, navigating through data and extended visualization.

Apart from recording information, land mobile mapping is highly relevant to scene understanding and classification of objects. Therefore, providing critical scene information either in real-time or after post-processing is critical. Identifying semantics and geometry of objects is essential part of the process. Nowadays, the enhancements in DL models, and the ever-increasing datasets, support a growing trend toward performing on-board data processing and scene understanding using the collected measurements from the mobile system. These include real-time detection, tracking, and semantic segmentation of both dynamic (e.g., pedestrians) and static (e.g., road markings or signs) objects in a scene, which could be part of the final deliverable.

In essence, since both MMS and mobile mapping services are expensive, the final deliverables can be tailored to the customer and the specific application.

16. Bathymetric Surveying

Bathymetric surveys are the sum of all acquisition technologies and processing methods to create a 3D map (or equivalently a DTM, Chapter 17) of areas covered by water, whether this might be sea, river or lake. Such surveys are measuring the water column above a large number of points and finally recreate the surface of the seabed and objects lying on it, including man-made structures and infrastructures.

Traditionally the primary output of bathymetric surveys are bathymetric charts. Printed bathymetric charts produce contour lines to show depth from a reference surface, usually mean seal level. These contours are known as isobaths. In the digital era, the primary output is GIS files, from which several visualizations can be produced, or directly used as input in GIS and other processing software for road design.

The technologies used to accurately measure the height of the water column are either vessel echo-sounding (sonar) or aerial LiDAR. Hydrographic surveys are using surface or submerged vessels as platforms, equipped with submerged sonar devices, hence the sound signal travels only in one medium. Aerial surveys are performed using airborne LiDAR sensors, where the Laser signal travels in two mediums, hence dealing with water refraction in post-processing is a necessity. Both implementations are in essence Mobile Mapping applications, whose fundamentals and supporting technologies are discussed in detail in Chapter 15 of this Volume.

According to Table 3-4, bathymetric surveys may be used in preliminary (1:2,000 - 1:5,000) and detailed design (1:1,000), when a bridge over a water body is necessary. Given that bathymetric surveys may only provide depth information,

Table 2-2 requires that the vertical accuracy expectancy for 1:1,000 surveys is 0.10 m RMSE for spot elevation or 0.13 m for contours, while for 1:2,000 surveys the requirements are 0.33 m and 0.25 m respectively. Unfortunately, only the latter can be partially fulfilled, with current technology and equipment.

16.1.Echo - Sounding

An echo-sounder is a system determining water depth based on sound reflected to sea bottom. The system consists of a sound transmitter and a receiver that picks up the reflected echo, electronic timing, and amplification equipment, along with a display for operational control and coverage verification. Echo-sounders may be attached to the hull of the vessel, towed from a surface vessel or onboard USVs (Figure 16-4). An echo-sounder sends a sound pulse into the water, the sound energy travels through the water to sea bottom where it is reflected towards the receiver, where it is timed and recorded. The principle is similar to the one used on LiDAR (Chapter 14), but LASER is replaced by sound due to better propagation characteristics in water (Figure 16-4).

Echo-sounding takes advantage of the fact that water is an excellent medium for sound propagation. As such, in water a sound pulse will bounce off a reflecting layer, returning to its source as an echo. The time between transmission of a sound and reception of its echo bounced from the sea bottom, can be used to determine the depth.

To use speed of sound to measure water depth, we need to know exactly how fast sound travels in the water medium. The speed of sound depends on water temperature, salinity, and pressure along the water column. If such environmental factors are monitored by sensors, then speed of sound can be adjusted, for precise depth estimation. The speed of sound in water varies between 1,400 and 1,570 m/s or roughly four times faster than air (Equation 16-1). It is extremely critical to accurately predict the speed of sound because the depth estimation depends directly on it. Given the variations in speed and their dependance on environmental factors, use of sensors to record such environmental factors is obligatory.

$$\textbf{Water Depth}\,[\mathbf{m}] = \frac{\textbf{\textit{Speed of sound}}\left[\frac{m}{s}\right] \times \textbf{\textit{Roundtrip Time}}[s]}{2} \qquad \text{Equation 16-1}$$

In Equation 16-1, the roundtrip time is divided by 2 to account for the two-way trip to the sea floor and back.

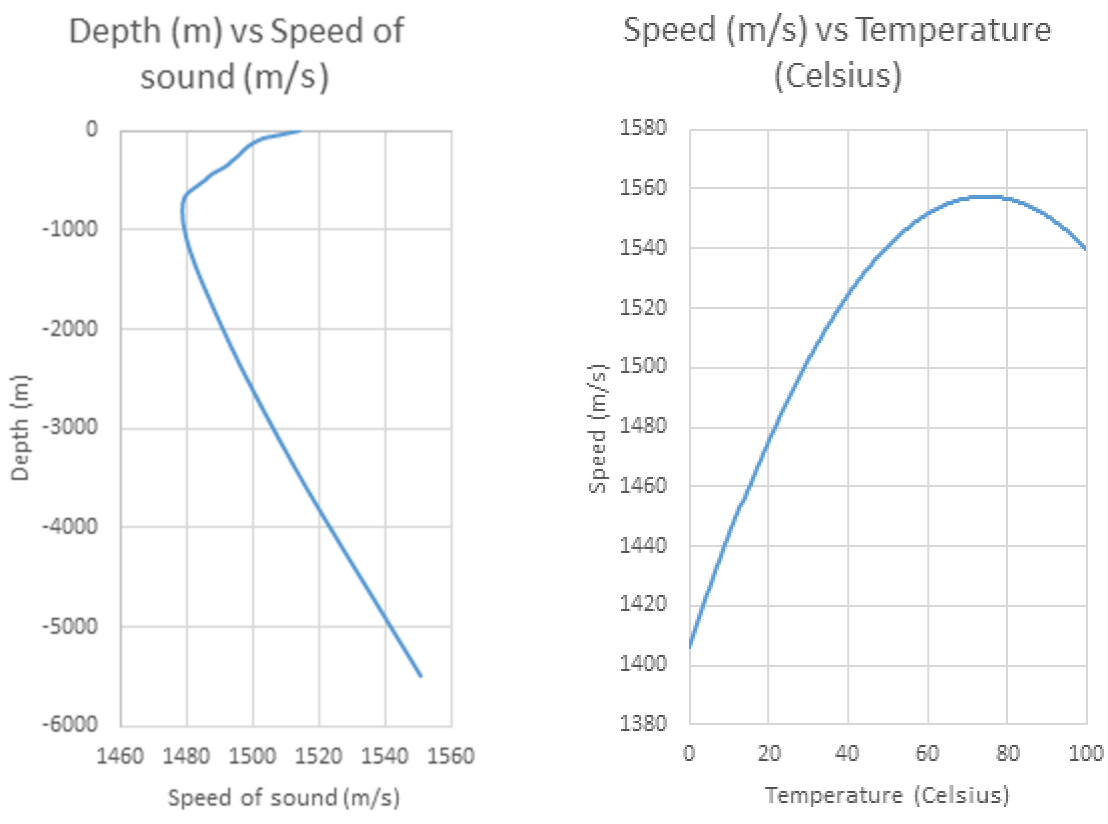


Figure 16-1 Sample of Sound Speed Variations at Different Depths and Temperatures

Sound speed is critical for such operations, hence shall be compensated appropriately. Use of environmental sensors for temperature, salinity, and pressure, which are the main factors affecting empirical equations (Kinsler, et al., 1982), is the most common way.

$$C(Z,T,S) = 1449.05 + T(4.57 - T(0.0521 - 0.00023T)) + (1.333 - T(0.0126 - 0.00009T))(S - 35) + D(Z) \quad \text{Equation 16-2}$$

where:

T = the temperature in degrees Celsius

S = the salinity in parts per thousand

Z = the depth in km

D(Z) ~ $16.3Z+0.18Z^2$

Given that the depths for bridge foundations will not exceed 100 m, the most important factors to take into consideration is temperature and salinity. The correction of sound speed can be done in transducer head (readings or salinity and temperature only on head) or in real time integration of the sound velocity profile. Although the later, is the optimal scenario, given the small depth for bridge foundations, the transducer head correction may also be used.

Uncertainties of such systems are not discussed in detail, nor in units by the vendors. Some technical sheets, refer that the product can meet IHO Special Order standards (Figure 16-2). Special Order accuracy standards for hydrography/bathymetry as established by the International Hydrographical Organization (IHO) in Monaco and disseminated in Special Publication No. 44 (S-44) (IHO, 2022), in 2020, is defined as 0.25 m plus a function of the depth of Equation 16-4. Given that the echo sounder survey accuracy (Figure 16-2) is 0.25 m or worse, it may only fulfil (Table 2-2) 1:2000 scale for contours, or 1:5,000 scale both contours and spot elevation, hence adequate only for preliminary design. Even if Exclusive Order standards are fulfilled, still 1:1,000 scale map vertical accuracies cannot be satisfied.

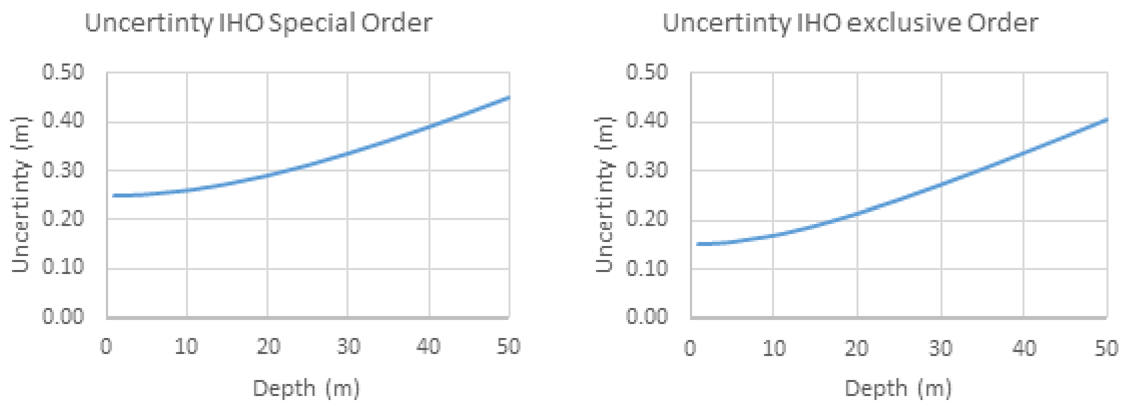


Figure 16-2 IHO Special and Exclusive Order Standards

Echo-sounders can be divided into two categories; Single-Beam Echo-Sounders (SBES), Multi-Beam Echo-Sounders (MBES) global (Wölfl, et al., 2019). Side Scan Sonars are also using technology based on sound echo but cannot be used to derive reliable depth information. They may only provide visual information related to bottom relief, and sometimes it is combined with SBES or MBES systems for better understanding sea bottom and structures or objects on it.

16.1.1. Single - Beam Echo Sounders

Single-Beam Echo-Sounders (SBES) are primarily used by hydrographical surveyors for measuring the depth of water directly beneath the echo sounder, measuring one point for every ping. This type of echo-sounder usually has the emitter and receiver (transducers) in a single enclosure. The sound is transmitted in a single beam of sound and records a single depth as a result of every measurement. Most systems wait for a pulse to come back before transmitting the next one. They usually record 20 - 30 points per second in shallow waters, while in deeper ones, the recording speed depends on the time sound needs to travel to the bottom and back.

The beam opening angle is important since the returned depth will be the average over the cone's base area. Beam angles themselves, vary between a few degrees to a few tens of degrees. A small beam angle means that only a small part of the bottom is recorded which supports better horizontal resolution of the echo sounder. The higher the sound frequency, the smaller the beam angle with similar transducer sizes. A smaller size at the same frequency indicates a larger beam angle.

The main advantage of a SBES system is its robust construction, small size, and low cost. The disadvantage is that it only measures single depths along the line sailed by the vessel, thus many dense parallel lines need to be recorded for covering an area. Usually areas covered by SBES, have varying density of points, in an irregular gridded pattern (Figure 16-3). It is also quite difficult to survey shallow waters, since the vessel needs to be directly above the surveyed points, which is not the case with MBES (see below).

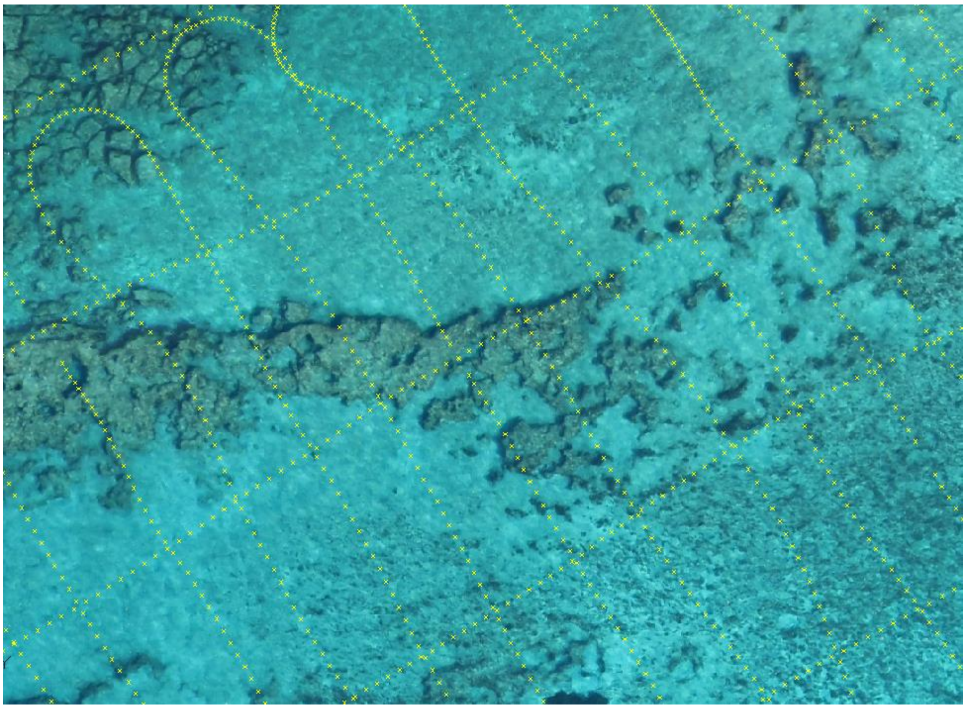


Figure 16-3 Sample of SBES Point Acquisition

In summary they may be used for bathymetric surveys, provided the density of points and specified coverage allow for such approach.

SBES characteristics:

- Cheap (relatively).
- Best used for sections on the sea bottom.
- Good for harbor and port surveys, or costal surveys, covering limited area.
- Ideal for small boats and harsh conditions.
- Depth range 1 cm to 6,000 m.
- Side scan option for some.

Stonex Nemo110, Unmanned Surface Vessel.

Source link: www.stonex.it

PHANTOM H-3000 Deep Ocean Engineering.

Source link: www.deepocean.com

Figure 16-4 Examples of USV from STONEX and Deep Ocean Engineering, which can be Used as Platforms for SBES, MBES, or even LiDAR Sensors

16.1.2. Multi - Beam Echo Sounders

MBES offer the hydrographical surveyors the ability to detect and record seabed depths along a swath beneath the survey vessel. To obtain those depths the transducer sends out a pulse of sound which is reflected of the bottom and received by an array of transducers. The system has a single transmitter and a number (usually 256) of receivers, which operates in medium or normal frequencies. The received beams are formed upon reception. The swath angle varies between 120 and 170 °, depending on the vendor. The bottom swath depends on the swath angle and water depth, usually being 3 to 25 times the water depth, depending on whether the system is optimized for shallow or deep waters.

Most MBES are ‘shallow water’ MBES with ranges between a few tens of meters up to a few hundreds of meters. Current MBES have beam angles between 0.5 and 5 °, which corresponds to the bottom area reflecting the echo. Depending on the implementation, the depths are measured in the so-called equi-angular or equi-distant mode. Similarly, to SBES, the ping rate depends mainly on the water depth being as high as 60 pings/second in shallow water.

MBES are generally hull mounted or pole mounted to a vessel. Most MBES can be operated in SBES mode. If more information is required, then water column data can be selected as an option. This allows the user to digitize the entire signal on the water column and thus allow detection of objects in it. Another option for many MBES is backscatter data. In this case not only the depth is measured but also the received signal strength. Different objects will give different amounts of reflection thus allowing discrimination between different material types on bottom.

MBES (Figure 16-5) can be mounted on pole or vessel hull, or in USVs. Subsea MBES can be mounted on ROVs or AUVs, although such underwater approaches are used for deep waters.

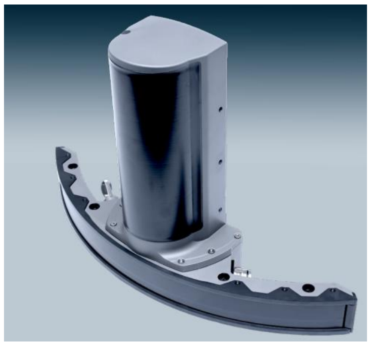
The main advantage of MBES is their ability to fully cover the area of interest, instead of acquiring a few spot heights and interpolating in between (Figure 16-3). MBES have the following properties:

SeaBat T20-P

Source link: www.teledynemarine.com

EM 2040P MKII transducer, front and rear view.

Source link: www.kongsberg.com



Norbit Winghead B44.

Source link: norbit.com

Figure 16-5 MBES Examples from Teledyne Marine, Kongsbesg Maritime and NORBIT

- Compact, lightweight (approximately 7 - 10 kg); stainless steel housing, urethane acoustic window
- Good for Passage surveys and General Hydrographic surveys.
- Detailed coverage over the Area Of Interest.
- Operating frequency varies from 0.5 - 700 kHz.
- Detectable depths from 0.5 - 11,000 m.
- Swath angle varies between 120 and 170 °.
- Effective beam widths of 0.5 to 5.0 °.

16.2.Aerial LiDAR Surveys

Aerial surveys with LiDAR can be used for bathymetry in coastal areas and depths up to 50 m depending on water visibility and turbidity. Since the infrared wavelength (e.g., 1,064 nm) is absorbed in water, LiDARs for bathymetry are using both infrared and green laser (e.g., 515 nm or 532 nm). Both wavelengths are used combined; the former wavelength to estimate level of water surface and the latter to estimate depth. Water refraction compensation must be performed during post processing. Since the first reflection (LiDAR return) corresponds to water surface and the last to the sea bottom, it is possible to calculate the water depth and corresponding water refraction angle deviation.

The depth penetration depends on water visibility and varies from 4 to 50 m. The density of points depends on the flying height and LiDAR equipment characteristics, but it is equivalent to land LiDAR surveys. Accuracy for sea bottom points is worse than the equivalent in land. LiDAR sensors for manned aircraft surveys are integrated equipment comprised of the LiDAR itself, an accurate positioning system and usually an imaging sensor. Software provided by the vendor, can process raw data from the LiDAR device. Post processing includes:

- Alignment of neighbouring flight lines.

- Full wavefront processing.
- Water refraction correction (not necessary in aerial LiDAR land surveys).
- Colourising of the point cloud if there is a camera on board.

Vendors provide accuracy information for their system as a whole and in all cases fulfil Special Order accuracy standards for hydrography/bathymetry as established by the International Hydrographical Organization (IHO) in Monaco and disseminated in Special Publication No. 44 (S-44) (IHO, 2022), in 2020. Given that the airborne LiDAR accuracy (Figure 16-2) is 0.25 m or worse, it may only fulfil (Table 2-2) 1:2,000 scale for contours, or 1:5,000 scale both contours and spot elevation, similarly to the echo sounding methods (see Section 16.1). In summary, they are as accurate as MBES, fulfilling IHO Special Order, but they have limitations in terms of depth and may not be efficient for small areas, such as surveying a bridge passage over a lake or a river.

On the other hand, aerial LIDAR surveys are excellent for extended coastal surveys, since they can record both land and sea bottom seamlessly, minimizing dead areas in the wave breaking zone and capturing both on the same flight, at the same time, using the same acquisition system rather than LiDAR for land and echo sounders for depth. Nevertheless, LiDAR depth penetration limitation may exclude them from some applications.

16.3. Accuracy Expectation for Bathymetry

Both technologies and corresponding procedures of echo sounders and LIDARs can only fulfil IHO Special Order accuracy, which is roughly but not strictly equivalent to vertical accuracy of 1:2,000 scale maps.

Since the vertical accuracy expectations for 1:1,000 scale maps cannot be fulfilled by either, detailed design shall suffice with data from either one of those. At this point it should be mentioned that since SBES cannot provide full and detailed coverage of the interest area, only MBES and LiDAR surveys with dense point coverage shall be used to extract contours and spot elevations for bathymetry.

Given that LiDAR aerial bathymetric surveys, conform to general aerial LiDAR methodologies and procedures as mentioned in Section 13.2, the remaining of this section, will be focused on echo sounder bathymetry (Jayaweera, et al., 2018).

16.4. Equipment

An SBES or MBES echo-sounding equipment consists of the following:

- Transmitter.
- Receiver.
- Oscillator.
- Electronic Timing and Amplification Equipment.
- Indicator or Graphic Recorder.
- Ship / Boat / Towed Vehicle.

When preparing for a bathymetric survey, one must ensure that the equipment used must be capable acquiring data that fulfill survey requirements. Therefore, an initial investigation of the total propagated uncertainties of all equipment, methods and corrections used to complete the survey must be included. The temporal and spatial influence of the medium, in which measurements take place, must also be considered in this initial uncertainty calculation. By this a priori calculation, in a specific scenario, it can be determined whether the instrumentation setup is sufficient for the requirements of the survey. If uncertainties cannot be calculated prior to the survey, an alternative methodology of describing the achieved uncertainties must be undertaken to verify that the requirements will be met.

Additionally, the equipment in use should be free of (systematic) errors which must be determined by regular calibration and certification as declared in Section 4.2 of this Volume. The use of calibrated equipment is the first step towards a quality control process. Entire system should be checked in real conditions (in situ) before surveying, as well as every time there is doubt about their performance. Maintaining a data-pack for key equipment and/or including the relevant information in a final report accompanying each survey is recommended (IHO, 2022).

16.5. Procedure

Organizations and companies undertaking bathymetric surveys, adopt standardized procedures for all stages of the process, from data collection to processing and verification, to ensure service quality and minimizes error risks. Their equipment must be calibrated on an annual basis. Using standardization in all procedures, it is possible to include regular checks and tests so that errors occurring in early stages, can be detected identified and properly treated. This is particularly important for errors that cannot be detected nor rectified afterwards.

Procedures may involve flow diagrams, that can be used for external auditing and standardization of final products. Integral part of the procedures are quality assurance checks in all steps of the process from preparation, data acquisition, methods, data post-processing and final production.

16.5.1. Depth Measurement and Geodetic Reference

MBES as a hydrographical survey tool has significant advantages over SBES in its ability to detect small objects and achieve 100 % consistent bottom coverage, in an efficient manner. It requires key ancillary equipment such as an appropriate motion and heading sensor (according to mobile mapping applications, Chapter 15), which must be properly integrated for correct operation. This adjustment needs to be done in any re-installation of the equipment in a new vessel. The ability to estimate sound velocity profiles through the water column is necessary to correct for the beam refraction. MBES with flat transducer arrays also require an accurate instantaneous measurement of the sound velocity at the transducer to enable correct beam steering to occur.

Users should be aware of the expected performance of the system and employ robust acquisition methodology to prove this before accepting the system as operational. Careful calibration of MBES is required at regular intervals thereafter. A reference surface such as an area of seafloor where repeatable measurements can be compared, should be used to verify repeatability of the system (Ora, et al., 2020), during the beginning and the end of the survey.

Inherent with the increased detail and coverage achieved with MBES is the ability to clearly see errors associated with incorrect lever arm and sensor offsets, time delays, sound velocity and excessive vessel motion. The ability to 'average' or 'smooth' out such errors in subsequent processing is often overestimated and misleading. Post-processing in such context should be avoided unless the magnitude of the change from raw to smoothed record is clearly described. Such errors should be stated and included in the final report for the calculation of the overall accuracy value accompanying the data (IHO, 2022).

Correlation to α vertical reference must be established, as in some cases the reference plane of the survey may differ from the geodetic reference plane, i.e., if the depths should be related to the lower tide of the surveyed area, to ensure sufficient under keel clearance, at shipping channels.

Also, tidal information should be applied in real time, if the collection period is long or the tidal effect is considerable. GNSS data are usually used to validate and correct such effects.

16.5.2. Positioning System Equipment

The depths measured by the SBES or MBES, need to be corrected for vessel movements and georeferenced. GNSS is widely used to fix vessel position during bathymetric/hydrographical surveys. The source of the differential corrections should be proven by comparison with a known survey control point or a network system. GNSS receivers should be configured to output positions in the desired datum.

In MBES systems, position system latency should also be determined and applied in the survey acquisition program. If possible, a dynamic check against a distinctive bottom target for which a known position has been derived should be undertaken, as this serves to reveal any latency or vessel/boat layback errors not otherwise detectable with a static check. Bottom targets should be located in shallow (i.e., less than 10 m) water to ensure the echo-sounder footprint and subsequent resolution of the target is comparable with the positioning system in use.

16.6. Data Processing

It is recommended that data processing be conducted using a dedicated hydrographic processing package that preserves data integrity. Modern packages offer almost complete flexibility and the potential to "manipulate" or overly "smooth" data. Surveyors should refer to the manufacturers' instructions accompanying survey processing packages and develop an appropriate methodology and standardized operational approach in relation to equipment and software used.

The post processing of hydrographical/bathymetric survey data should include georeferencing, removal of gross errors and noise, to produce a "cleaned" data set (Figure 16-6) for further processing or for the generation of required products (e.g., sounding sheets) for subsequent analysis. In case of MBES, it is also necessary to ensure seamless depths across swath lines and possible cross check lines.

Where possible, standard nautical hydrographical symbology should be used on survey sheets; in particular, the standard convention of displaying depths as meters and decimeters where the decimeter is shown in subscript form should be followed (e.g., 56 instead of 5.6) (Ora, et al., 2020).

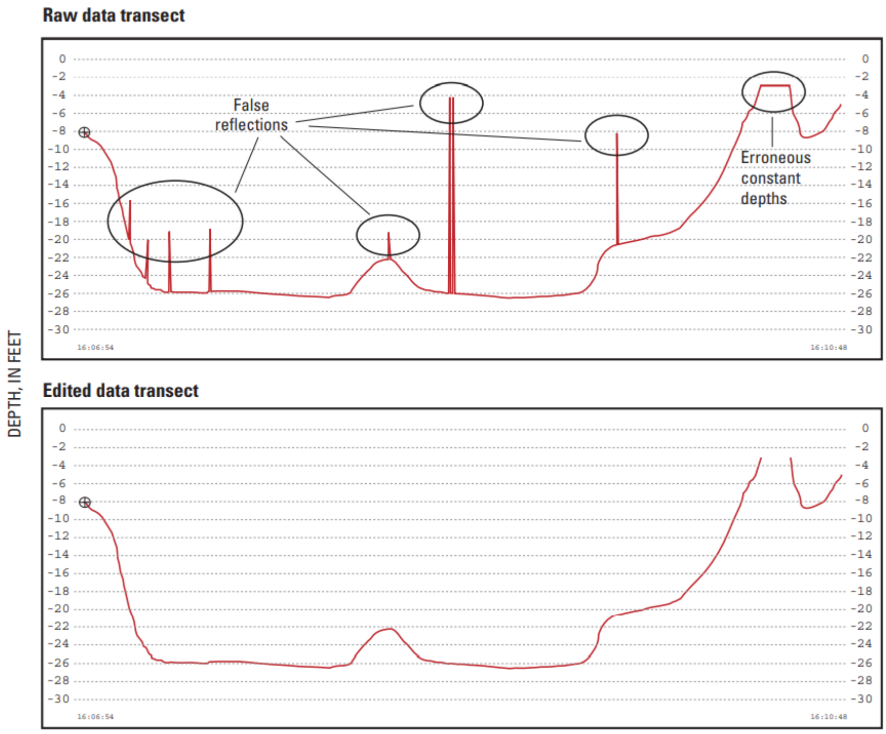


Figure 16-6 Raw and Edited Transect Data (Wilson & Richards, 2006)

The bathymetric surveys should be conducted by personnel with appropriate qualifications and/or certifications (FIG, 2010)

16.7.Accuracy and Precision

To achieve the required quality (accuracy and precision) three important fields are affecting it: Material, Procedures, and Personnel. All of them are essential for a quality-controlled survey. Quality control is not just about figures and computations, but rather a combination of all factors affecting the survey.

The final horizontal and vertical accuracy of a bathymetric survey depends on several factors:

- the type and quality of the depth measurement system (echos);
- the resolution of measured depths (echos);
- system calibration and alignment (AUV);
- vertical and horizontal reference datum accuracy (dWL);
- vessel draft errors, platform stability, vessel velocity (AUV);
- and vessel positioning system (AUV);
- adopted collection strategy (AUV, dWL);
- weather and environmental conditions (echos);

all of which are merged under Equation 16-3. In depths up to 2,000 m the 2-sigma error (95 % confidence level) degrades to 7 - 8 m (Zwolak, et al., 2020).

$$\boldsymbol{\sigma_d^2 = \sigma_{dAUV}^2 + \sigma_{d\,echos}^2 + \sigma_{dWL}^2} \quad \text{Equation 16-3}$$

16.7.1. Positioning System Equipment

The accuracy standards for hydrography/bathymetry are established by the International Hydrographical Organization (IHO) in Monaco and disseminated in Special Publication No. 44 (S-44) (IHO, 2022), in 2020. To accommodate in a systematic manner different accuracy requirements for areas to be surveyed, IHO describes five orders of survey. These include, from loose to strict:

- Order 2.
- Order 1b.
- Order 1a.
- Special Order.
- Exclusive Order.

Order 2 surveys are intended for areas where the depth of water is such that a general depiction of the bottom is considered adequate. Such surveys are conducted in areas that are deeper than 200 m. At least a 5 % evenly distributed bathymetric coverage is required for the survey area.

Order 1b is intended for areas where the types of surface vessels expected to transit the area is such that a general depiction of the bottom is considered adequate. As a minimum, an evenly distributed bathymetric coverage of 5 % is required for the survey area. This order of survey is only recommended where under keel clearance is considered not to be an issue.

Order 1a order is intended for areas where features on the bottom may become a concern for the type of surface traffic expected to transit the area but where the under-keel clearance is considered not to be critical. A 100 % feature search is required to detect features of a specified size. Under keel, clearance becomes less critical as depth increases, so the size of the feature to be detected increases with depth such as areas deeper than 40 m. Examples surveys are coastal waters, harbors, berthing areas, fairways and channels.

Special Order survey is intended for those areas where under keel clearance is critical. Therefore, 100 % feature search and 100 % bathymetric coverage are required and the size of the features to be detected by this search is deliberately more demanding than for Order 1a. Areas requiring such surveys are: berthing areas, harbors, and critical areas of fairways and shipping channels. Special Order surveys are accomplished by the use of closely spaced lines in conjunction with side scan sonar, multi-transducer arrays or high-resolution multi-beam echo-sounders to obtain 100 % bottom search.

Exclusive order hydrographic surveys are an extension of IHO Special Order with more stringent uncertainty and data coverage requirements. Their use is intended to be restricted to shallow water areas, such as harbors, berthing areas and critical areas of fairways and channels, or in specific critical areas with minimum under keel clearance and bottom characteristics are potentially hazardous to vessels. A 200 % feature search and a 200 %

bathymetric coverage are required, which can be accomplished by adequately overlapping collection or by acquiring more than one independent datasets within a survey. The size of features to be detected is deliberately more demanding than for Special Order.

16.7.2. Definitions and Quality Metrics

The most common metrics involved in a hydrographic survey are:

Bathymetric coverage: Extent to which an area has been surveyed using a systematic method of measuring the depth and is based on the combination of the survey pattern and the range of the survey instruments.

Feature: Any object, whether natural or manmade, which is distinct from the surrounding area.

Feature detection: Ability of a system to detect features of a defined size.

Feature search: Extent to which an area has been surveyed using a systematic method of identifying features.

Total horizontal uncertainty (THU): Component of total propagated uncertainty (TPU) calculated in the horizontal dimension. THU is a two-dimensional quantity with all contributing horizontal measurement uncertainties included.

Total propagated uncertainty (TPU): Three-dimensional uncertainty with all contributing measurement uncertainties included.

Total vertical uncertainty (TVU): Component of total propagated uncertainty (TPU) calculated in the vertical dimension. TVU is a one-dimensional quantity with all contributing vertical measurement uncertainties included.

In recognition that there are two factors affecting the vertical uncertainty, one depth-dependent and another one depth-independent, is used to compute the maximum allowable vertical measurement uncertainty.

$$\boldsymbol{TVU_{max}}(\mathbf{d}) = \sqrt{\boldsymbol{a}^2 + (\boldsymbol{b} \times \boldsymbol{d})^2} \qquad \textbf{Equation 16-4}$$

The parameters "a" and "b", together with the depth "d", have to be introduced in Equation 16-4 to calculate the maximum allowable TVU.

Uncertainty: Estimate the range of values within which the true value of a measurement is expected to lie as defined within a particular confidence level.

Table 16-1 Brief Bathymetric Standards for Safety of Navigation Hydrographic Surveys

Order	THU	TVU	Feature Detection	Bathymetric Coverage
2	20 m + 10 % depth	a = 1.0 m b = 0.023	Not Specified	5 %

Order	THU	TVU	Feature Detection	Bathymetric Coverage
1b	5 m + 5 % depth	a = 0.5 m b = 0.013	Not Specified	5 %
1a	5 m + 5 % depth	a = 0.5 m b = 0.013	Cubic features > 2 m in depths up to 40 m; 10 % of depth beyond 40 m	≤ 100 %
Special	2 m	a = 0.25 m b = 0.0075	Cubic features > 1 m	100 %
Exclusive	1 m	a = 0.15 m b = 0.0075	Cubic features > 0.5 m	200 %

16.7.3. Bathymetric Standards

The IOH Standards for Hydrographic Surveys (IHO, 2022), describe in detail all standards, along with accuracy and precision for all orders of survey. A brief description is given in Table 16-1, but other approaches depending on the specific requirements of a construction site can be defined in a similar manner.

16.8. Delivery

Survey deliverables refer to all data, reports, and products associated with a hydrographical/bathymetric survey that will be submitted prior or during a construction. Given that the bathymetric surveys fall under the category of DTM, all requirements are similar to those described in Section 17.5 of this Volume.

The required Client hydrographical/bathymetric survey deliverables include:

- Project Survey Reports.
- Digital Data.
- Metadata.

Optionally:

- Hardcopies of charts and maps.
- Raw data acquired.
- Data georeferenced, before noise removal.

The delivered contours should be edited to comply with cartographic rules (Figure 16-7).

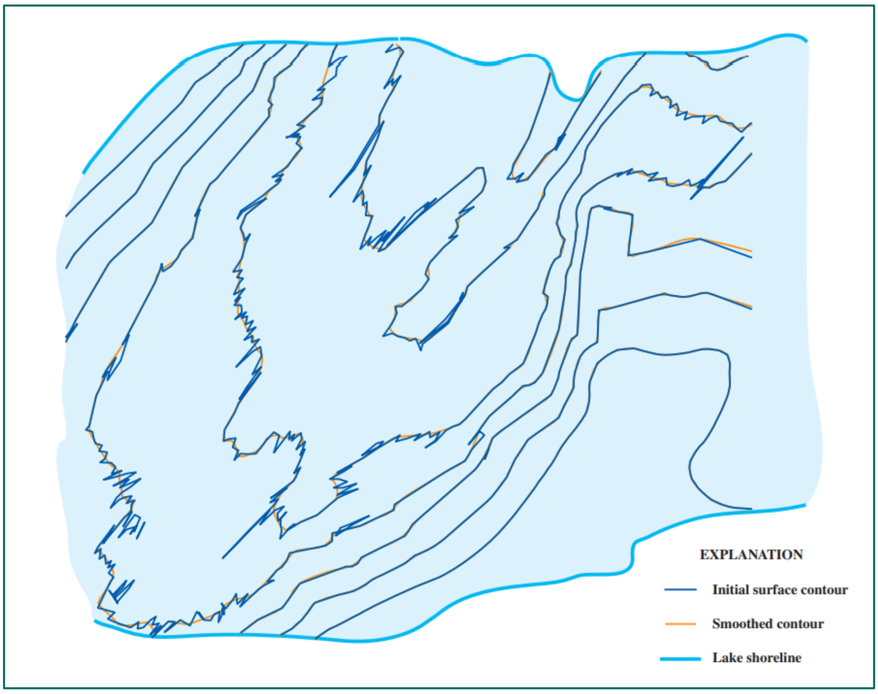


Figure 16-7 Representation of Initial Surface Contours and Cartographically Smoothed Contours (Wilson & Richards, 2006)

17. Digital Terrain Modelling

DTM surveys aim at obtaining the representation of the surface of the Earth. More specifically, the purpose of DTM survey is to determine and compute the DTM of the project's area, which is a topographic model of the Earth's surface that can be manipulated by computer programs. Furthermore, DTM is an elevation surface representing the surface of the Earth referenced to a given coordinate system. Development of DTM enables the aspect of elevation and adds 3D capabilities to design projects; it also assists in procedures such as designing road alignments, culverts and buildings placement etc. Generally speaking, terrain models have an essential impact in the process of road and construction related projects. The data provided as input for the development of a DTM are collected mainly via aerial, i.e., photogrammetric and LiDAR, procedures, as well as traditional field survey methods such as TPS, GNSS etc.

17.1.DTM Features

When a Digital Terrain Modelling Survey is realized, it is important to consider the method employed to collect the necessary data. The selection of the method ensures the accurate, rapid and rigorous completion of the survey. Generally speaking, DTM is constructed using aerial methods (photogrammetric/LiDAR), while ground survey methods (TPS, GNSS) help in data acquisition that otherwise would be unreachable. When areas share characteristics of dense vegetation or, for any reason, unreachable areas, aerial and/or LiDAR methods can secure the collection of data. On the other hand, when a DTM survey has large scale requirements close to constructions and/or buildings that pose obstacles on the aerial data collection, ground survey methods can be adopted to secure these data. Moreover, simple aerial photogrammetric methods cannot be applied into areas of dense vegetation; in this case only LiDAR or ground TPS/GNSS surveys may be of help.

17.1.1. General

DTMs are the main products being produced during a DTM Survey. Such models can be delivered using photogrammetry, aerial LiDAR (Chapter 13), or by field survey methods using TPS or GNSS. Generally, a DTM consists of spot shots, and breaklines (Figure 17-1):

- Spot shots are mass points with a labeled elevation and are added in areas where the terrain is relatively flat or uniformly sloping. Generally, spot shots are placed at the natural highs and lows of the terrain and relatively flat or uniform areas.
- Breaklines are linear strings compiled along the breaks or changes of grade in the terrain, identifying terrain discontinuities such as ridges, toe of slopes or any abrupt change in the existing surface.

Abundant and sufficient density of measured terrain elevations is very important regarding accurate DTM computation. Moreover, given that automatic photogrammetric methods capture the DSM, it is common that such data include altitude information about the highest viewing surface. As such, in the case that specific terrain areas of the project cannot be accessible by the photogrammetric methods (such as under bridges, trees, high grass or other features), then a void will be left in the data. If deemed necessary and decided that the void created needs to be filled then supplemental field survey measurements shall take place (MnDOT, 2007).

DTMs are modeled using the Triangle Irregular Network (TIN) model, which is generally computed given spot shots and breaklines. In TIN model, points collected are connected in order to form a network of triangles covering the surface of the project area. Therefore, a TIN is a representation of a continuous surface consisting entirely of triangular facets.

Figure 17-1 Typical DTM Features Collection

TINs are computed using the Delaunay triangulation method. A Delaunay triangulation for a given set *P* of discrete points in a general position, is a triangulation *DT(P)* such that no point in *P* is inside the circumcircle of any triangle in *DT(P)*. Delaunay triangulations maximize the minimum of all the angles of the triangles in the triangulation; they tend to avoid sliver triangles. They are widely used for modeling terrain or other objects given a point cloud, since such triangulation gives a nice set of triangles to use as facets in the model. In particular, the Delaunay triangulation avoids narrow triangles (as they have large circumcircles compared to their area).

On the other hand, narrow triangles can be found in real-world terrain modelling. Thus, the need for constrained Delaunay triangulation arises: a generalization of the Delaunay triangulation that forces certain required segments into the triangulation as edges, unlike the Delaunay triangulation itself which is based purely on the position of a given set of vertices without regard to how they should be connected by edges. These edges in terrain modeling are known as breaklines which force the triangle legs to tie into the breakline of the discontinuity. DTM surveys must give certain weight to the correct capturing and modelling of breaklines. For example, a ditch bottom or terrain irregularity left undefined by a breakline will triangulate across the discontinuity and show the terrain as flat in those areas.

Finally, given the computed TIN file, a variety of products can be calculated based on an interpolation of the TIN data (MnDOT, 2007): contours (Figure 17-1), cross-sections, volumes etc. are the most common derived products. Table 17-1 summarizes specifications and procedures regarding DTM creation.

Table 17-1 DTM Procedures and Specifications by Scale

Specification	Units	Scale			
		1:5,000	1:2,000	1:1,000	1:500
Maximum distance between mass points	m	50	20	10	5
Maximum distance between consecutive points of the same breakline	m	50	20	10	5
Maximum distance between cross sections	m	100	40	20	10
Maximum void zone area that needs not to be covered by DTM.	m^2	10,000	1,600	400	100

17.1.2. Spot Shots

Data points identify the XYZ (3D) coordinates of a location and are collected in profile or grid mode. They are obtained at locations of significant grade change. Recent advances in photogrammetric methods, e.g., SfM, allow collecting mass data points over a grid of very small spacing of e.g., 0.05 m. On the other hand, in cases where a field survey is required, mass points cannot be further from each other than 10 m for large scales (Table 17-1). Depending on the surveyed area they should be collected in a near-grid, or cross-section, form in the field, with their in-between distance not exceeding 10 m. Random points may be also used when necessary to identify unique terrain situations, such as a steep irregular hill side with varied vegetation, pits etc.

17.1.3. Breaklines

Breaklines represent the abrupt change of grade in the existing surface. In order for the breaklines to be included in the process of DTM's development they must be created in a 3D environment where their elevation has been assigned. Therefore, breaklines must be represented in the form of line strings in the 3D space. The distance between the data points of a single breakline linestring shall not exceed 10 m for large scales (Table 17-1).

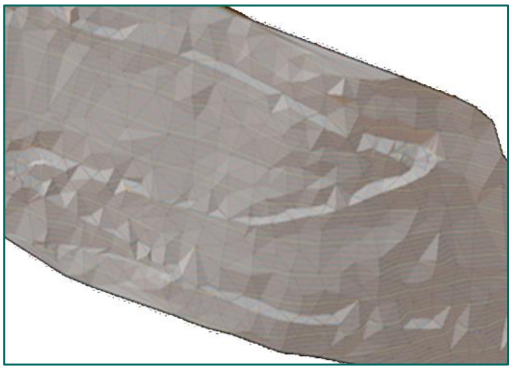



Figure 17-2 DTM With (a) and Without (b) Breaklines

Correct computation of a DTM is largely dependent on breaklines' correct placement. Poor placement or absence of breaklines, results in non-accurate DTM calculation. For example, Figure 17-2 (a) illustrates the DTM constructed with the employment of breaklines, while Figure 17-2 (b) illustrates the same area with the DTM produced without breaklines, where it is revealed that their absence leads to misinterpreted road surfaces, side slopes etc.

The major elements for which breakline data collection shall be performed are, (some of which are illustrated in the section of Figure 17-3):

- Roadway centerlines.
- Inner and Outer edges of shoulders.
- Curb gutter lines.
- Top of curbs and of slopes.
- Toe of ditch slopes.
- Ditch bottoms.
- Toe of slopes.
- Top of slopes.
- Embarkments.
- Crowns.
- Retaining Walls.
- Edges of roadway pavement.
- Sidewalks.
- Drainage structures.
- Concrete Items.

Other abrupt changes or "breaks" in the terrain are not limited to the above catalog. Any other terrain break found during a DTM survey shall be also surveyed and included as a breakline in the DTM creation.

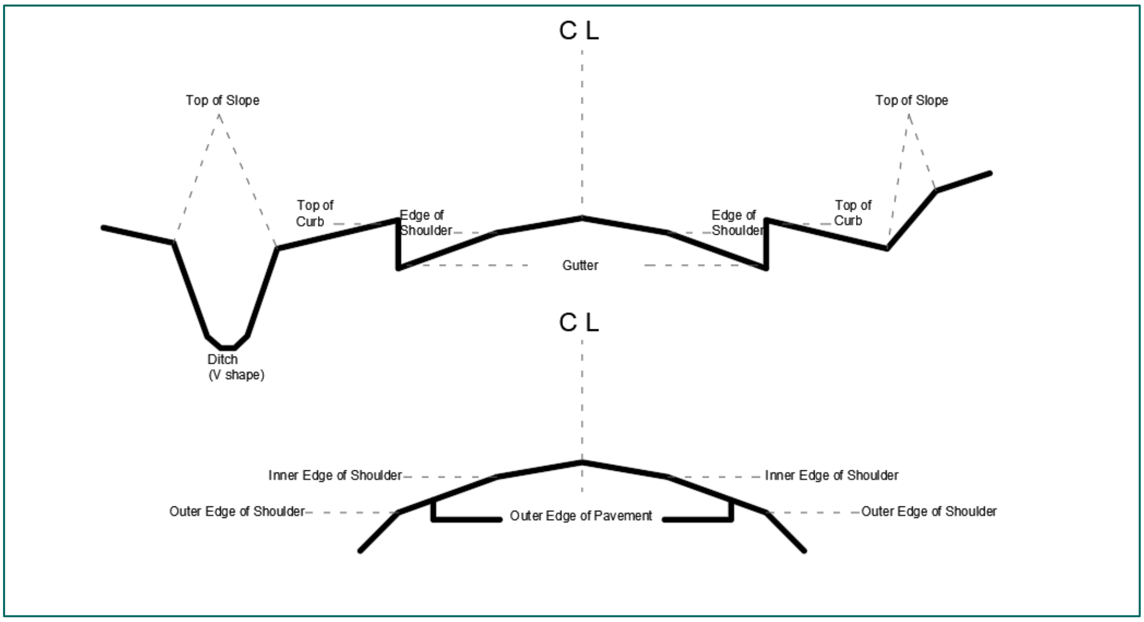


Figure 17-3 Breaklines Placement - Typical Section

Finally, obscure breaklines are also used in order to cover void zones in circumstances where elevations of data points cannot or have not been obtained.

17.2. Terrestrial DTM Data Collection

Several variables should be considered during a terrestrial DTM survey. To start with, it must be assured that the distribution of the points collected on the ground corresponds and correctly models its actual surface. Moreover, all features essential for the correct DTM computation such as breaklines, spot shots etc., have to be collected.

Correct spacing of random points and correct placement of breaklines are critical to accurately model physical sites with a DTM. Random points at all local minima and maxima within the project's area shall be collected; local minimum or maximum is a location within the modeled site that is at a low or high elevation relative to neighboring points. Additionally, random points on a grid or cross - sections should be collected throughout the site so that the distance from one random point to another in the grid is about equal and not exceeding 10 m for large scales (Table 17-1).

When the terrain surface is described by abrupt changes and discontinuities such as ridges, toe of slopes, top and bottom of ditches, roadway crowns, the edges of roadway pavement etc. breaklines shall be surveyed so as to accurately represent the surface. In general, breaklines should not cross one another. If breaklines intersect each other, problems may be experienced triangulating the surface (Section 17.4). Crossing breaklines should have the same elevation in their intersection point, since a surface cannot contain two points at the same X, Y and have a different Z.

Breaklines shall be surveyed as 3D objects with an adequate number of surveyed points so as to sufficiently define the breakline feature in 3D, with spacing between sampled points of the same breakline, no more than 10 m for large scales (Table 17-1). Data collection shall always include sufficient number of points representing as accurately as possible the breakline in 3D.

17.3.Aerial DTM Data Collection

Digital surveying data from photogrammetry have become the most frequently used data by planners and designers, especially in creating DTM necessary in road planning and design processes. One source of this data is photogrammetric modeling derived from aerial photography or aerial LiDAR flights.

Modern mapping aircrafts equipped with GNSS antennas and cameras can capture nadiral photos employed in DTM surveys. Contemporary photogrammetric DSM acquisition is done almost fully automatically exploiting SfM and MVS techniques, instead of manual stereoplotting used during the analytical era of photogrammetry in the past. Details are given in Chapter 13.

An example of a photogrammetric DSM acquisition is displayed in Figure 17-4 (a) where a DSM of 5 cm GSD is rendered using the hillshade technique, allowing to visualize the vegetation that is included into the respective model, which is also visible in the corresponding orthophotomosaic of Figure 17-4 (b).

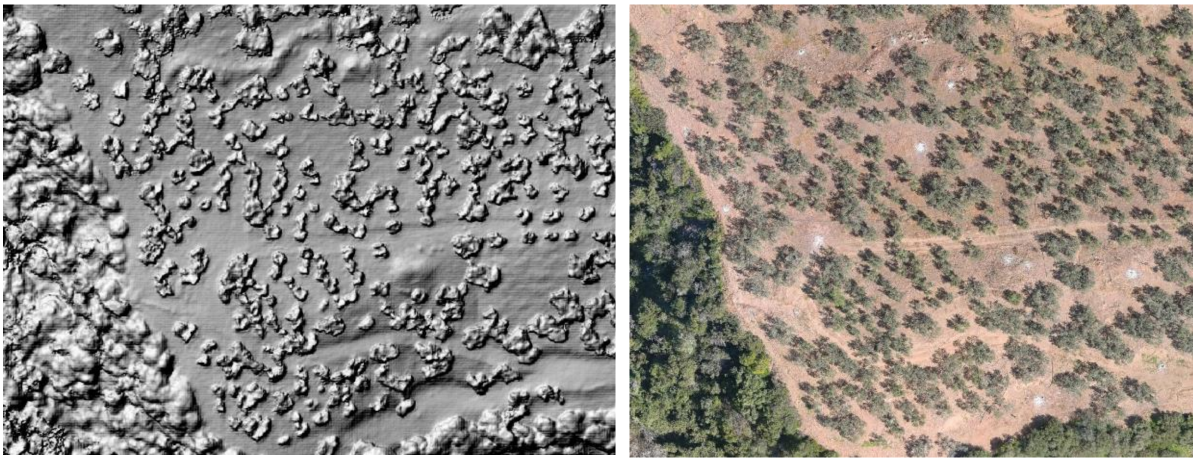


Figure 17-4 (a) Digital Surface Model from Photogrammetry and (b) Othophotomosaic

In order to produce DTM from automatically created DSM data, vegetation/structures included in the DSM shall be always removed, using the following methods:

- Manually applying internal boundaries with obscure breaklines around them.
- Employing appropriate automatic classification algorithms to identify ground point data by removing vegetation and structures (Figure 17-5). This procedure can be achieved by dividing the automatically created point cloud into cells of a certain size, and then selecting the lowest point giving a first approximation of the actual terrain model. Then, new points are iteratively added into the model providing that they fulfill certain geometric requirements regarding the current terrain approximation (e.g., distance, angles etc.).

While the first, manual, approach is rather inefficient, it produces valid and accurate results. On the other hand, point classification algorithms are extremely efficient in terms of human resources, but highly dependent on the employed parameter values. Therefore, such automatic approaches shall only be used with caution and results shall be checked and verified by an experienced user.

Figure 17-5 (a) Raw Point Cloud and (b) Automatically Identified Ground Point Cloud Data

In any case, interpolation has to be applied within the automatically or manually created void zone. Void zones that have to be covered by DTM, e.g., zones covered with vegetation, shall not have areas of values greater than the ones shown in Table 17-1; otherwise, one of the following alternatives should be followed to complete the void zone:

- Terrestrial DTM survey with traditional TPS and GNSS methods.
- Manual steroplotting to acquire terrain data if possible.

Moreover, breaklines have to be also manually extracted from the DSM, or, by stereoplotting, and included on the final DTM. Still, even under these limitations, photogrammetric DTM surveys are much more efficient regarding traditional field (terrestrial) DTM surveys; therefore, they should be preferred, given the general specifications illustrated in Table 2-1, Table 3-1 and Table 3-4.

On the other hand, LiDAR aerial surveys technology, have the unique characteristic advantage of automatic vegetation removal in LiDAR data, as a higher equipment cost. Nevertheless, if true DTM is needed, man-made structures should be marked and removed from the acquired data. Still, the advantage of automatic vegetation removal makes LiDAR aerial survey ideal for automatic DTM acquisition in highly forested/vegetated areas.

17.4.DTM Construction

DTM is constructed with the aim of software packages implementing Delaunay and constrained Delaunay triangulation. Regardless of the software package used, four main elements need to be defined for DTM construction:

- Mass points.
- Breaklines.
- Inner Boundaries.
- Outer Boundaries.

Among them, the first two have been extensively discussed in the previous sections; regarding the inner and outer boundaries, these are used in general to hide areas where DTM can be created, nevertheless its form as created by the software is not valid. For example, regarding inner boundaries, their purpose is to describe void areas, that need not to be covered by DTM, e.g., building footprints; as such inner boundaries serve as obscure breaklines.

Outer boundaries are used to prevent the creation of DTM in areas where its representation is not valid, near the outer boundary of the surveyed area: Delaunay triangulation used for DTM creation forms triangles in the entire convex hull that is shaped by the set of points and breaklines used in the triangulation. However, this is not always valid since triangles formed near the outer convex hull area tend to be sliver as shown in Figure 17-6 (b), resulting in highly erroneous contour lines on the area; on the other hand, Figure 17-6 (a) depicting the same area with an outer boundary, displays DTM information only on areas where it is actually valid.

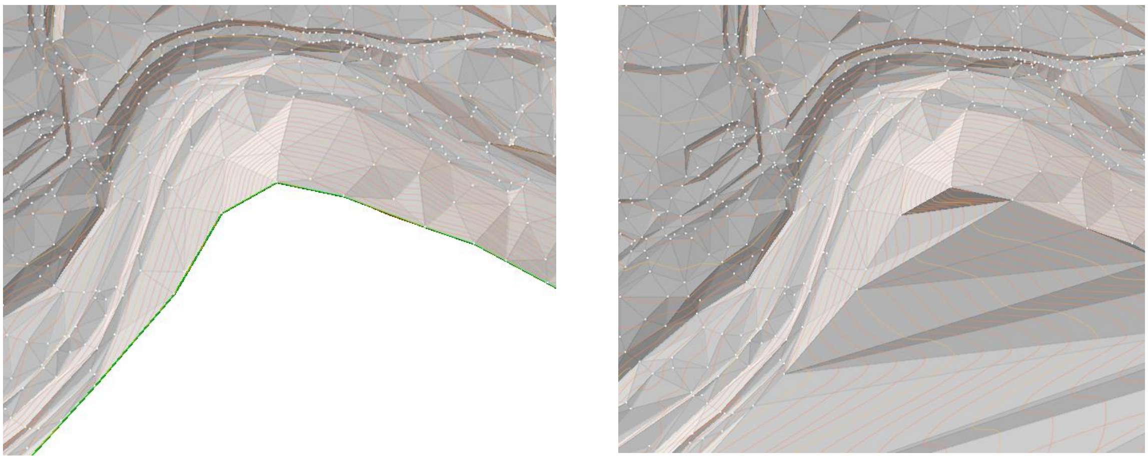


Figure 17-6 DTM Creation (a) With Outer Boundary and (b) Without Outer Boundary

Finally, both types of boundaries, outer and inner, serve also as breaklines in the DTM creation. All these elements have to be represented in 3D, i.e., each spot or linestring point shall carry 3D information.

As such, the following rules apply:

- All elements used in DTM creation shall be represented in 3D.

- When appropriate inner boundaries shall be used to remove void areas that need not be covered by the DTM.
- When appropriate outer boundaries shall be used to remove triangles from the outer DTM area.
- Final DTM shall fully represent the terrain elevations in all the covered area.

Before DTM construction, regardless of the software used, the following tests should take place to ensure that all elements are correctly used in the DTM construction:

- **Crossing Breaklines**: If breaklines intersect each other in 2D, problems will be experienced triangulating the surface, since in the point of 2D intersection each breakline provides its own elevation for the respective point (using linear interpolation between the previous and the next breakline linestring point). However, surfaces represented by Delaunay triangulation cannot contain two points at the same X, Y having different Z values. Therefore, crossing breaklines should share the same elevation at the intersection point, by adding a common vertex on both breaklines.
- **Spot points being close to one another**, on a user-defined threshold shall be checked for consistency. Such points are not usually collected in the field and tend to be included in the survey by mistake, e.g., point surveyed with incorrect instrument height, which has been surveyed correctly again without deleting the previous one.
- **Points close to breakline edges not included in the same or another breakline,** on a user-defined threshold. If a single spot point is close to a breakline it may be an indication that it should be part of the breakline. Such errors tend to form sliver triangles in the triangulation, resulting on false terrain interpretation.

After completing above tests and initial DTM creation is performed, contours should be drawn, or visualized according to software capabilities, e.g., hillshading, draw cross sections etc. A visual check shall be performed by the surveyor for discontinuities spikes and outliers in the constructed DTM. In these cases, after the identification of such data, they have to either be excluded from the triangulation, be edited to so that the correct elevation be assigned, reform breaklines and boundaries etc. Swapping triangles shall not be used; instead, a breakline shall be used to change the triangles' order. This is due to the need for reproduction, i.e., creating the DTM again by its basic elements, without the need for manual editing. The above process shall be repeated until no errors can be identified.

17.5.Delivery

DTM delivery includes all DTM elements described in previous sections, along with the final triangles constructed by the respective software. Therefore, final DTM delivery shall include in 3D:

- Mass/spot points.
- Breaklines.
- Inner Boundaries.
- Outer Boundaries.
- Output Triangles.
- Contours with interval according to scale as described in Table 2-2.

Above elements should be delivered in vector, e.g., DXF, DWG, DGN, formats. Suitable GIS formats (e.g., shapefile, geopackage etc.) may be also used for this purpose. Sections 6.3 and 6.4 provides information about CADD/vector data delivery. Final DTM shall be delivered in ESRI TIN format.

Interpolated DEM produced from the constructed DTM according to the GSD size and file formats of Section 6.2 may be also delivered, especially in the case of photogrammetrically created models.

18. Cadastral for Road Projects

According to (KSA Cabinet Decision No. 54/1437, 2015), landowners in KSA are entitled to receive compensation for the parts truncated for the planning or for the determination and implementation of the main roads' networks. Therefore, the need for land surveys determining public and private properties in the land arises.

Land surveying refers to the initial instalment or reinstatement of boundaries and monuments regarding real properties, based on documents, historical evidence, actual field conditions and existing standards of practice. Land Surveys document the policies, standards, and procedures regarding boundary control of road projects (MnDOT, 2007).

Land Surveys should provide accurate locations of horizontal and vertical positions of roads, buildings, public lands, private boundaries, Right-of-Way (RoW) and RoW plats and other future developments. A *plat* is a cadastral map, drawn to scale, showing the divisions of a piece of land. Plats show the distance and bearing between land section lot corners, sometimes including topographic or vegetation information.

18.1. Organization

All tasks of land surveying are guided and supervised by all concerned sections and client's departments. The organization aims at (MnDOT, 2007):

- Finalizing RoW plats.
- Reviewing all land survey related documents including files from KSA land registration system.
- To support in decision making regarding the appropriate localization land lot corners and highway, RoW lines, especially in complicated or debatable conditions. Special cases may require consulting KSA Ministry of Justice.
- To maintain a liaison with all sections of KSA Ministry of Justice, MoTLS and MoMRAH, in addition to other governmental agencies on every land surveying aspect.

18.2. Public Land Surveys

Locating and certifying public land corners near highway properties and RoW are of great importance regarding both project's owner and the public. All data used in the purchase of highway property for public use, such as legal descriptions, highway plats and maps are based on these government land corners. Therefore, their significance rises: if one of these is somehow incorrectly positioned, all properties described or referenced from this corner could be open to future boundary disputes and/or costly litigation (MnDOT, 2007). Such positions should be perpetually marked and re-established if marks are destroyed. Concerned authorities should perpetuate and replace all known section and quarter section corners that are destroyed or obliterated by construction, reconstruction or maintenance of a public highway.

18.3. Private Boundary Corners

Private boundary corners are usually corners marking subdivision plats, registered land surveys, and/or bounds surveys. The location, re-establishment and perpetuation of these corners are a necessity to highway right-of-way acquisition and monumentation (MnDOT, 2007).

18.4. Equipment

The equipment used for land surveys includes:

- Total Station (TPS).
- GNSS using RTK/PPK techniques.
- Other lower cost equipment such as tapes, Laser Distance Measures, accessories regarding TPS and GNSS instruments such as poles, bolts, prisms, and targets.

18.5. Field Research

A thorough research will take place to determine all public and private property boundaries that fall within, touch or by any means impact the area of highway construction. The inclusion of all necessary land corners for legal land ties is of utmost importance so as to eliminate future legal issues.

After all property – related documents have been acquired, if any, their information must be verified and supplemented on the ground. Ground checks are made for possible gaps, overlaps and discrepancies in the survey data that would have an effect on the concerned authority's RoW or the value of the expropriated land.

Steps that have to be taken include the verification of record evidence on the ground, reconnaissance of occupation lines and roads and interviews with property land owners and local residents (MnDOT, 2007).

18.5.1. Verification of Record Evidence

The first phase of field research is to locate land corners monuments that are known and actually exist on the field. Such monuments are based on records that they do exist subject to local changes that might indicate that the monument has been destroyed, e.g., deep road excavation after the date the record evidence was collected. Missing land corners will be determined so as to be further reestablished with the help of other features.

18.5.2. Reconnaissance of Occupation Lines and Roads

Subject to the lack of a property corner, a field survey should be made so as to retrieve occupation lines, roads, and property corners that could be used to substantiate the missing corner; such elements used to implement a missing corner must be fixed to the control survey. If lot corner monuments are missing or do not seem to fit the occupation lines, it should be noted on the field blue line of the plat or on the certificate of survey. Occupation lines may have to be tied in to analyze and verify the platting (MnDOT, 2007).

18.5.3. Interviews - Property Owners & Local Residents

Interviews with property owners and local residents are essential, providing valuable information about damaged/unable to be found land corners, as well as, information about the real boundaries of parcels within the project area. Before the interview, the interviewer should become well-informed about the extent of the project and the approximate dates of various phases of the planned construction. The interviewer should also understand the limits on information he/she can give to property owners and local residents. The interviews should include questions about the land, specifically (NYSDOT, 2009):

- Request for help locating property corners and monuments.
- Ask for information about possible property disputes, feuds, or legal proceedings over property boundaries in the vicinity.
- Request for help locating sections, quarter, meander, or judicial monuments in the area.

Interviewers, should make short written notes of names, dates and significant facts during the interview; upon its completion complete notes should be written. If a statement relating to the verification of a corner monument is needed, they should be drafted and signed during the interview, by the interviewer and the property owner.

18.5.4. Land Corners Identification

Subject to the lack of records, legal documents, and cadastral system regarding land being affected by the project under design, a cadastral should be issued, documenting any public and private land. Land corners should be determined over implemented boundaries, e.g., fencing.

In the absence of implemented boundaries. neighboring land owners should agree on site on the boundary of the respective properties; these land corners should be subsequently surveyed with GNSS/TPS surveys, permanently marked and monumented. Statements relating to the verification of such a corner should be drafted and signed during the interview, by the interviewer and all involved property owners.

18.6. RoW Base Map

The RoW base map is the planimetric map used in the RoW process to the acquisition stage (MnDOT, 2007). A new RoW base map is usually needed when a project requires additional RoW or urban development has occurred on existing projects. Every RoW base map should include all natural and manmade features acquired during location surveys in the site, as well as imagery background if available. Specifically, the following characteristics should be mandatorily included:

- Existing roadways with their name and number.
- All property corners found during the record research and field survey.
- Block and other property lines.

- Existing right of way lines, of any kind, with dimensions so that any proposed right of way which will tie into the existing right of way can be computed.
- Manholes, catch basins, utility pedestals and valves. Avoid showing linear connections of such utility structures.
- Location of culverts without indicating details.
- Wells (used or unused) located within 10 m of the RoW.
- Underground fuel tanks and sewer systems including septic tanks, outlets and drain fields.
- Existing fence, boundaries and land corners.

Annotations should be included in the RoW base map, regarding both road alignment data (PI, PC, PT, radiuses, curve lengths etc.), as well as previously defined characteristics such as names of street, country roads and railroads, rivers, streams and building information (type of structure, number of floors).

Finally, when a large RoW map extends on multiple sheets, an index map shall be at the right side of each sheet.

18.7.RoW Plats

Generally speaking, all RoW to be acquired / and all land to be expropriated, is to be by reference to an acquisition plat. The plat defines the RoW access, any temporary and permanent easements, that the road authority is acquiring. Property ownerships and parcels being acquired are shown graphically annotated with a parcel code.

The name of the owner of the property and the area of each type of interest being acquired from each owner is shown in tabular form, along with the respective parcel code.

All acquisition plats are referenced to the KSA-GRF17; plat boundary corners are also computed. Basic data on conversion factors, ties to Public Land Survey lines and corners and other data will be found in the plat folder or survey report for the project. The data needed to compute the coordinates of plat’s boundary corners are:

- The RoW map with the construction limits and the proposed RoW, preferably in the form of polygon(s) enclosing the expropriated area.
- Parcel data (boundary coordinates) in the form of polygons.
- Details about the alignment, e.g., the coordinate values for the proposed alignment points.
- Data regarding subdivision plats affected by plat boundary, including coordinates of found corners.
- A data file for RoW boundary curve data.

The number and location of the boundary corners for each plat are determined from the RoW map.

18.8.Staking RoW Boundaries

For purposes other than platting and large-scale monumentation, the marking and staking for RoW boundaries includes the following:

- RoW Staking for Viewing or Appraising.
- RoW Staking at the Request of Adjoining Property Owners.
- RoW Staking for Private Land Surveyors.

18.8.1. RoW Staking for Viewing or Appraising

Staking for viewing or appraising is performed when then concerned RoW engineer requests a review. Survey crew obtains the currently valid RoW map and uses it for guidance. Land corners are not permanently staked; temporally stakes are placed with an accuracy of ± 0.3 m (MnDOT, 2007).

18.8.2. RoW Staking at the Request of Adjoining Property Owners

Property owners of adjoined plots may demand RoW staking to assure that they do not place improvements on land that does not belong to them. Posts and signs are usually placed in this type of staking; it may also require relocating and re-establishing the centerline used for the purchase of the RoW (MnDOT, 2007).

18.8.3. RoW Staking for Private Land Surveyors

This type of staking is done by providing a coordinate file of control, alignment and RoW boundary points for staking by the private land surveyor.

19. Construction Surveys and AMG

Prior to the construction activity of any project, the design information has to be transferred from the plan to the ground. This is accomplished in general, by construction surveys and staking. Construction surveys are employed for any construction project, from roads to buildings, tunnels to hydraulic works and underwater structures. Construction surveys are extremely useful and important since all construction phases are based on their output. Establishment of control network, project setting out, and pre-construction survey are prerequisites to the actual construction work. Construction survey in general continues through all construction work stages: for example, in highway construction, construction surveys guide all works from the initial earthwork to the lighting pole establishment, and even after the end of it so as to produce precise as build drawings.

Survey works for highways and related constructions such as bridges etc., are divided into three main categories: pre-construction, construction, and post-construction. Complete details of construction surveys for roads, bridges, parking lots and buildings can be found in following Sections 19.1 to 19.5 of this Volume.

Standard equipment used in most survey operations is employed for construction surveys:

- TPS.
- GNSS.
- Leveling.
- TLS.
- Other lower cost equipment such as tapes, Laser Distance Measures, accessories regarding TPS and GNSS instruments such as poles, bolts, prisms, and targets.

All equipment used in construction surveys should be checked and verified and that the equipment maintains it's proper calibration before actual construction/staking work begins. Calibration instructions and specifications given in Section 4.2 of this code also apply here, in addition to the specific calibration guidelines provided in the calibration section of each instrument type of this code, e.g., Section 7.3 regarding TPS.

19.1.Road Corridors

Road construction surveys refer to the location of alignments, slopes and other road basic features on the ground to guide the construction and achieve a result as much as possible close to what the designer has drawn in the respective drawings. As for all construction surveys, highway construction surveys involve pre-construction, construction, and post-construction phases as presented in the following paragraphs.

19.1.1. Project Set Out

The project's owner provides initial control on which the construction survey will be tied. Project control could be established with appropriate monuments during project's design phase with the initial control surveys. Contractor/surveyor is responsible for the recovery of the initial control survey by locating on site the appropriate monuments.

After control recovery, it shall be checked whether it is good and follows the accuracies of the respective order. If the control is good, contractor/surveyor determines whether additional control is needed for construction purposes and is responsible for staking/establishing it. If not, the field control will have to be reset. Project's owner and contractor/surveyor should review all control survey data and decide the best method of accomplishing a proper control. The Contractor is required to submit this staking information to the project's owner, which should also check its accuracy.

19.1.2. Pre-Construction

Prior to commencing the actual construction survey work, surveyors assigned to a construction project should carefully examine the plans and special requirements regarding project's construction. The project engineer should be aware of any significant error or deficiency that will come into the attention of the surveyor regarding the project under examination. Project engineer will then take all the necessary steps to resolve them (IDOT, 2015). Client/project owner should be informed on these and express its opinion.

A thorough project's review of this type not only allows to detect errors and deficiencies, and also allows the surveyor to be familiarized with the project and be better prepared to plan his operations when actual construction works begin. Preconstruction plan review, note preparation, miscellaneous computations, and fieldwork are essential for a smooth operating construction project.

Harmonious relations between all stakeholders involved in project construction, are necessary for a smooth-operating project. Good communications between all involved roles, i.e., contractor, engineer, surveyor, and inspectors, help to accomplish this goal. Moreover, it is highly important that the surveyor establish a working relationship with the contractor and any other supervisor, making the surveyor's job much easier and greatly reduce the possibility of errors.

As such, pre-construction procedures include the establishment of a good relationship between all involved personnel. Towards this goal, prior to the beginning of construction activities, a conference shall be held by the Consultant with the Contractor and his supervisory personnel as well as the Client's concerned engineering personnel. This meeting is of particular importance to the construction surveyor since he must plan and organize his duties to meet contractor's planned sequence of works so that there will be no delays or disruptions in its work. During this, the contractor and its personnel should be made aware of the importance of properly maintaining stations and benchmarks during all construction works (IDOT, 2015).

19.1.3. Construction

During the construction phase, survey crew stakes all essential road elements guiding earthwork, paving and final roadway surface position. Further details regarding constructions surveys are given in following paragraphs.

19.1.3.1. Work initiation

Initiating staking work, survey team should take the initiative and confer with the Contractor to determine the portions of the work to be staked first. It is essential that both parties should mutually agree upon:

- The lines and grades desired.
- The clearances required for construction equipment.
- Other matters related to the layout work.

The survey team should arrange their work so they will always have sufficient stakes set ahead so the Contractor will not be delayed in starting the work.

Given that markings of various road design points, are typically obliterated during the clearing and grubbing phase, it is helpful to establish reference points outside the clearing limits. Reference points should be set at least 3 to 5 meters behind the uphill clearing limits. On the average, reference points should be set at least every 70 to 100 meters. Typically, reference points are placed at points where the center line alignment can be easily re-established, such as points of curvature.

Before setting stakes, survey lines must be well established and verified. All control points should be used to reference new points outside the area of construction; new points should tie into PIs, PCs and PTs. Construction benchmarks should be set and checked before stakes are set.

All levels taken while setting stakes shall be tied to benchmarks before stake elevations are used. Once stakes are set, they shall be guarded with lath and high visibility flagging. The lath shall clearly identify the stake and its use in order to avoid confusion (IDOT, 2015).

19.1.3.2. Work - checking

Regardless of the employed staking method, work should be checked as much as possible to avoid the possibility of errors. Data provided by the respective study such as grade elevation, curve data, etc., should be checked before being used. All measurements, level notes and computed distances should be frequently verified. Checking should be performed with the plans in hand instead of relying on surveyor’s memory.

19.1.3.3. Recording

Proper project construction requires that the field layout work will be done accurately. Field notes should be kept according to uniform practices and conform, as a minimum, to neatness, legibility, clarity, completeness, permanence, and accuracy. Field notes recording shall also follow the instructions given in Section 4.4.5.

It is crucial that field books containing all necessary elements such as grades, sketches, line ties, benchmarks, and other data, are ready in advance. Lack of preparation will result in referring to the plans often for layout information which will further result in delays and inconvenience. Independent checks over notes and sketches should be applied. Grade elevations at the intervals required, survey line ties, benchmarks, curve data and any other data required for frequent use should be contained in the books (IDOT, 2015).

Survey books shall keep a complete legible record of all stakes set. It shall also include the following:

- The description, dates set, coordinates and elevations of all new benchmarks, as well as the new ties for all points.
- Any grade or other changes from the original design should be included and carefully described.

19.1.3.4. Staking methodology

A standard method of staking should be followed but may be varied to meet topographical conditions, type of construction, equipment used and the Contractor's preference. This is a convenience to contractors who work in more than one region, and it also reduces the chance of confusion and misunderstanding between engineers in the field and the Client representative (IDOT, 2015).

Contemporary geomatics and technology advances, lead to automated construction, with staking requiring much less effort for the field surveyor. Modern instruments including GNSS, total station, and robotic instruments, are used in staking of not only single points rather than the entire proposed project's model. While this significantly reduces field calculation, it is still essential that the field crew must properly stake all elements to the ground. Office staff should periodically check instruments to confirm that the currently valid model or points are loaded into the respective instrument (IDOT, 2015).

19.1.3.5. Curves

The curve data listed in the plans should be carefully checked, including the PIs and the PTs. In setting stakes, usually the following steps are required (IDOT, 2015):

- Setups should be preferably made on PI having as a foresight and backsight another PI or PT. The tangent distance shall be measured accurately in each direction and hubs shall be set at the PC and PT of the curve. PC's station number is the one of PI minus the tangent distance. PT's station is the one of the PC plus the respective curve length.
- Before running curves, tables of stations, deflection angles and chord lengths shall be established and verified. After the PC and PT are established, set up takes place over PC proceeding to run stakes in the curve.

On several occasions, it is necessary to run curves in the opposite direction to that of the stationing. On long curves with obstructed view, it is necessary to turn at one or more points on the curve or curves to be set out with coordinate values.

Curves should close at most within 0.075 m error per 300 m of length; the closure error should be proportionally distributed over sufficient length. On curves of high radiuses having distances from the mid-point of curve to the PI, i.e., external distance, of 0.6 m or less, it is faster to run the curves by tangent offsets. In any other case, coordinate values of curve will be used to set out the curve.

Considerable difficulty can be encountered in running in the curves if the intersection angles do not check and new curve data must be calculated.

19.1.3.6. Recording - original ground levels of the project

With the end of the initial staking out, consultant and contractor shall perform a joint survey to record ground levels of the road corridor. To estimate the quantities of the earthwork for payments for roads, cross sections shall be taken at 25 m intervals; infrastructure projects shall use 25 m grid intervals for the same reason. Volume calculations shall be performed after computing the respective DTM.

19.1.3.7. Borrow pits

Setting stakes and getting cross-sectioning for borrow pits is crucial for accurate volume calculations: it should be done carefully with more than the ordinary accuracy so as to avoid confusion and possibility of dispute with the Contractor over the volumes involved.

Volume calculations for borrow pits can be performed using stakes over cross-sections set referencing a base line (and its parallel line on the opposite pit side), as described in (IDOT, 2015). In the case where the borrow pit is furnished by the concerned authorities, it should be staked before construction starts (for avoiding the Contractor to encroach upon private properties). On the other case, where the borrow pit is furnished by the Contractor, upon obtaining the necessary approval/licenses, surveyor shall establish the appropriate stakes and take the necessary initial cross sections. When stakes are used for borrow-pit volume calculation, and pits are adjacent to the right-of-way, it is suitable to use the road centerline as a baseline. The long dimension of the pit will be parallel to the baseline. In every case, baseline must be preserved or be able to be re-established until all work is finished. Base lines should always be straight lines regardless of the shape of the borrow pit, and reference points remaining on the ground after borrowing works are completed (IDOT, 2015). In this case, volume calculation can be performed directly by the delta elevation measurements taken on stakes, before and after earth borrowing.

Borrow pit volume calculation can be also performed by standard DTM procedures, which is more frequent in recent years. Specifically, any suitable method as the ones described in Chapter 17, can be used to acquire the initial and final (after borrowing) DTM and then use them to perform volume calculations with traditional methods (cross - sectioning) or any other method provided by contemporary software.

It should be noted here that in the case where a borrow pit is subject to overflow and the contractor suspends work for considerable duration, the pit should be cross sectioned immediately after work stops. In the case where an overflow occurs, the pit must again be cross sectioned prior to work starting as alluvial deposit may appreciably affect the measured quantities (IDOT, 2015).

19.1.3.8. Grading

Usually, three sets of stakes are used for controlling a construction contract. These include (IDOT, 2015):

- Right-of-Way, Control and Structure.
- Preliminary Grade.
- Finish Grade.

Prior to the staking operations, the respective method should be agreed with contractor. It should be chosen to provide the necessary accuracy for each operation, noted in the field book and given to the contractor in writing. Before starting dirt work operations, original ground elevations should be spot checked for accuracy.

After the earthwork is roughed in, a line of stakes is required, down the centerline of the roadway, providing the completed crown grade. After the roadway is built, being as close to grade as possible using the previous stakes, line and paving stakes should be set at 10 m intervals. Even closer intervals are required on tight horizontal or vertical curves. Stakes used for this purpose should be of metal of sufficient length to penetrate the grade far enough that the movement of equipment will not cause variations once the grade is established. Sub-base and pavement can be built from these paving stakes (IDOT, 2015).

Before earthwork operations, it may be necessary to place slope stakes to define the toe of the slopes for ditches and/or fill areas. These stakes should indicate cuts or fills for roadway and ditch, station location, offset distance and rate of slopes from the stake to the toe of slope.

A noticeable marker should be used at balance points, representing the intersection point of side slope and the existing ground level.

The curve data and typical sections shown on the plans should be reviewed for superelevation rates, superelevation limits, transition lengths and points of rotation. In some instances, this superelevation may create drainage problems, especially in wide pavements posed over in flat terrain. Any doubt regarding pavement superelevation should be reviewed with the constructor, the designer, and the Client (IDOT, 2015).

19.1.3.9. Culverts

Before staking out a culvert, it should be verified on site that the location as shown on the plans will fit the channel to the best advantage. In case of any doubt regarding culvert relocation or skew angle adjustment, constructor, designer, and client should be informed so as to examine whether culvert's location and characteristics should be changed.

All entrance culverts should be set to match the roadway ditch, both in line and grade. Culvert entrances are staked by two stakes on the centerline of the culvert barrel. On these stakes the cut to the ditch flow line should be marked. If headwalls are built, the top elevation of the two headwalls should be made parallel to the grade of the shoulder, even though the gradient of the ditch is not the same as that of the pavement (IDOT, 2015).

The culvert itself is staked by the centerline of its barrel, placing a stake on the centerline not closer than 1.5 m outside of each headwall. Stakes should include nails providing the respective line. Stake should be marked with the cut to flow line, measured from its top. The plan length should be always checked for accuracy.

Sometimes, after the forms are built, it is essential to set the elevation to be used for the top of headwalls. The elevations on the forms at which to set the chamfer should be provided to the Contractor. The tops of the headwalls must be parallel to the grade of the centerline of the roadbed (IDOT, 2015).

19.1.3.10. Pavement

Prior to the actual pavement staking, field books should be prepared, all computed grades shown on the plans should be verified, whereas any calculated grade for other points should be computed and verified. Given that new pavement is tied into existing pavement, all relative elevations should be checked.

The following data should be included in the field books, additionally to the elevations (IDOT, 2015):

- Elevations of each edge of pavement on superelevated curves and on superelevated transitions at ends of curves, at 10 m intervals.
- Ties to all survey line control points, PCs, PTs and benchmark elevations and locations.
- Tables of curve deflection angles and chords.
- Tables of offset from survey line to form stake line when required.

Pavement stakeout is done with stringline operations. The process of setting the stringline includes metal stakes to be set at 10 meters intervals along both or one side of the roadway. Setting stakes on both sides of the road, requires the utilization of a construction equipment with automatic slope control sensors attached on both the right and left sides of the machine. On the other hand, one side setting stakes, requires the use of machine with the sensors attached to its one side. The metal stakes are usually 60 cm long and are set to the hubs for both dual lane and single lane construction machines. On superelevated sections and ramps the interval of the metal stakes shall be set to 5 meters to achieve greater accuracy and precision (Lee, 2007).

The location and position of each stringline on the roadway shall be evaluated separately because the conditions of each section of the road may differ due to superelevation, crowns, and offsets. Additionally, each stringline must be set at a constant distance, from the roadway centerline (or a theoretical edge of the pavement). Each string line must also be suspended at a constant height above the plane passing through the lower corners of the proposed slab (Lee, 2007).

19.1.4. Post Construction / As - Built

Purpose of as built surveys is to check whether the actual construction has been completed as expected according to the design of horizontal alignment, vertical alignment, and cross-sections. As such, in this stage of survey works, it is confirmed that the construction complies with the design. All road elements from the utilities to the road drainage should be horizontally and vertically located after construction and put together in an As-built Survey. The final output of this survey is sent to the concerned authority so that it can be compared to the approved plans for the project.

As built survey includes, but is not limited to, all final built locations of structures, utilities, manholes, valves, storm drains, catch basins, curb and gutter, pavement, sign structures, light poles, traffic signals, etc.

Final measurements for constructed elements should be made preferably concurrently with construction operations, resulting in greater accuracy and reliability. For items that can only be checked after construction is completed, final survey will be performed afterwards.

Original plans must be updated to show any modifications and/or additions that were made during construction phase. In this case, corrections, changes and additions shall be explicitly noted on the final plan sheets.

The following list provides some of the information that must be field checked, corrected and added to the original plan sheets (IDOT, 2015):

- Horizontal and vertical control.
- Location, dimensions, and elevations of drainage structures.
- Changes in typical sections.
- Horizontal alignment.
- Profile grade.
- All underground units (cable, conduits, pipe, etc.).

The as-built locations of project items, including elevations shall be tied to the employed datum KSA-GRF17 and KSA-GRV14, unless otherwise explicitly stated in the respective contracts.

19.1.5. Accuracy

Construction surveys vary on a project-by-project basis. In addition, the use of automatic machine construction guidance is quickly advancing and will eventually be the norm rather than the exception. Any error occurred in staking may result in work that needs to be repeated and even lead to demolishing and repeating construction. Thus, the survey team should instruct any concerned authority to notify them at once if at any time a grade stake is possibly in error; this will provide the survey team the ability to check the elevation before work advances to a high degree.

Regarding additional control surveys that were set during the construction to augment the original project control, they should follow the same standards that were used on the original survey, following the accuracies provided in Chapters 7, 8 and 10 for control surveys. PIs, PTs, PCs and other staked alignment points shall also follow the accuracies provided for at least 3rd order control surveys. Base station set for machine control should also follow the same standards required of the original project control (IDOT, 2015).

The final surface of the earthwork shall be staked according to the profiles, slopes, cross sections, and superelevation provided in the respective design study with height tolerances of ± 30 mm. The levels of the upper earthwork surface of the road and railway works must be configured according to the elevations, longitudinal and transverse slopes provided by the study. The height tolerances in terms of the theoretical ones provided in the study, must not show deviations greater than 20 mm.

According to previous discussion the upper level of the earthwork may be staked with conventional GNSS RTK survey, using an appropriate benchmark as base withing the construction site. Final pavement shall be staked with TPS achieving higher accuracies.

19.1.6. Note Forms

Field notes in predefined forms for construction survey should be filled during the construction survey. These notes should contain stakeout listings for all surveyed elements with the actual staked positions noted in them.

19.2. Bridges

19.2.1. Project Set Out

The project's owner provides initial control on which the bridge survey will be tied. After control recovery, it shall be checked whether it is good and follows the accuracies of the respective order. If the control is good, contractor/surveyor determines whether additional control is needed for construction purposes and is responsible for staking/establishing it. If not, the field control will have to be reset. Project's owner and contractor/surveyor should review all control survey data and decide the best method of accomplishing a proper control.

Control points recovered, shall be checked of following the accuracies of the respective order, be re-establish if necessary, and used to add additional control. Bridge tolerances are much tighter than tolerances used for simple road construction. Therefore, project control checking is more time consuming.

19.2.2. Pre-Construction

The guidelines posed in Section 19.1.2 works for highways – road projects should be also followed here. Moreover, contractor should be contacted about where equipment and materials are to be stored, to avoid staking and establishing control points on this site.

Before staking a bridge, plan dimensions and elevations should be checked. Checking the elevations of the footing's bottom, is extremely important. A DTM survey should be performed beneath the bridge for the scope of profiling the edges and centerline of the roadway, or the wadi breaklines, to check them against the new structure's elevation. More detailed DTM information will be collected in the areas of the footings for earthwork calculations and contractor's payment. If required, a bathymetric survey shall be performed.

19.2.3. Construction

All aspects of construction staking mentioned in Section 19.1.3 of this chapter, are also applicable in Bridge Construction Survey. Additional measures and instructions required, are described in the following paragraphs.

19.2.3.1. General

Prior to the actual starting of construction operations, staking, checking, and referencing should be made regarding the entire structure. Stakes set during the construction are going to be used many times, from staking the footing excavation until the entire bridge construction is finally completed. As the construction work advances, several construction elements will obstruct visibility between stakes; this fact should be considered when positioning initial stakes and references. As such, having a few extra staked points is always welcomed. For this purpose, many stakes placed outside the actual construction area shall be used: at least three stakes on each line shall be set each way from the site. All stake's elevations shall be verified. Stakes shall be ensured that are protected, well referenced, and identified.

Positive control points should be placed on each pier and abutment so that during the construction of the bridge's cone embankments, instrument checks can be easily made detecting any movement.

Each stake and elevations given to the Contractor should be recorded. Each bridge feature should be sketched along with the placed stakes, their references, and respective distances. Staking diagrams and information should be shared with the Contractor to avoid possible future disputes (IDOT, 2015).

A third independent party should check all the layout work.

19.2.3.2. Triangulation

Given that high precision methods are required in bridge construction works typically involving long structures, abutments and piers' location should be measured with TPS employing triangulation methods, or any other precise measurement system (IDOT, 2015). Levels shall also be used for leveling purposes.

When locations are triangulated, especially in long bridges, only concrete monuments, or large stakes set deep and cut off near ground level will be used. Length measurements shall be always made employing with TPS employing EDM units.

When possible, intersection lines should be set for each pier at an angle of 45 degrees with the baseline; baseline should extend both sides from the centerline of the bridge. The intersection lines should be run out to points above high water on both sides of the wadis so that the locations can be set when the low ground is flooded. Angles should be set using repetition and should be checked by measurements (IDOT, 2015).

The general guidelines of Chapters 7 and 8 regarding horizontal and vertical control surveys shall be employed here.

19.2.3.3. Locating centerline

Bridge structure's centerline is not always identical to the one of the roadways. At least two PIs or PTs taken from the road plans or survey notes, should be established, and tied permanently, in each direction from the bridge. Scope of this process is to assure that the intersection angle in both directions from the bridge is right. In the case that PI or PT in each direction are not visible from the bridge, a new PT should be established on each side of the bridge, as close as possible to the initial PI or PT. A permanent hub should be placed on the centerline, close to the bridge, without obstructing Contractor's works. Contractor should be instructed to keep equipment and materials clear between these hubs. A permanent foresight should be set on the centerline of the bridge as high as the ground permits. It should be possible to set centerline from either side of the stream/wadi (IDOT, 2015).

Hubs on centerline of bearing or back of abutment and on the centerline of each pier should be established. Heavy stakes with nails for lines should be used for these hubs. Permanent benchmarks should be placed as close as possible to the bridge. Original benchmarks should be checked before establishing benches at the bridge site. New benchmarks should be transferred to permanent concrete or piling on the structure, and then be used for the remaining work. Temporary benchmarks should not be placed on newly constructed embankments since they may settle (IDOT, 2015).

19.2.3.4. Staking abutments

In staking the abutments, an allowance must be made for any anticipated deflection of the abutments so that the span length after deflection will be as originally described by the respective study. Deflection can be in theory computed. However, it can be uncertain due to the exact environmental settings on the backfill and footings. It is therefore assumed that the backfill pressure will transfer the top of a closed, or a concrete pile, abutment horizontally 5 mm for each meter of height measured from bridge seat to the footing's bottom. For open abutments, the same assumed movement is 2.5 mm per meter. If bridge is single spanned, and a deflection of abutments correction is necessary, the total deflection of both abutments should be added to the original span length shown on the plans (IDOT, 2015).

Abutment's skew angle should be set on the hub that is set on the centerline of bearing or the back of abutment with the centerline of structure. On this skewed line, hubs should be set as follows:

- One close to the bridge.
- Two at distances of 60 and 120 meters, or as far as possible from the bridge.

Skew angle should be checked by repetition before proceeding (IDOT, 2015).

19.2.3.5. Staking piers

Skew angle for piers is set in the exact same way as for abutments setting additional hubs in each direction. Surveyor should consider that the centerline of bearing and centerline of pier are not always the same. Pier's vertical alignment should be monitored with a Total Station during concrete placement (IDOT, 2015).

19.2.3.6. Staking footings

During footing staking, the elevation of the bottom as measured on site should be compared with the respective plans. If a difference of more than 30 cm is found, surveyor should consult with contractor, project's owner etc. so as to adjust design as needed. When the neat line forms of the footing are in place, the top of the footing should be established by setting nails with an instrument at convenient points around the footing. When footings are too deep to set elevations directly, turns may be established by measuring down to a nail from a point of known elevation (IDOT, 2015).

19.2.3.7. Miscellaneous elevations

All elevations should be set with levels and/or TPS. After the forms for either an abutment seat or pier cap have been built, grade points for the bridge seat elevations should be set with the respective instrument. Elevations at tops and wings should also be set with an instrument. Seat elevations should be checked after the concrete is placed. Bridge seat elevations should be checked by subtracting the deck thickness, minimum fillet, beam and bearing heights from the finished deck elevations before laying out the bridge seat elevations in the field. On steel truss spans supported by falsework, it is essential that each panel point of support be set at the exact camber elevation before any connections are made.

19.2.3.8. Wadi bridges

The structural elements of the bridge falling underwater will be set out from the ground control points and temporary control points should be fixed on the cofferdam for construction of each structure. These temporary control points should be frequently re-checked, at least before each construction stage, to confirm that the particular control has not been moved horizontally and vertically.

Guard stakes should be placed at each hub and the layout should be marked so that no confusion may result in. In several cases a low water and a high-water baseline should be established. Baselines above low water elevation can be usually placed near to the bridge site.

The elevations of the bottom of the footing as shown on the plans should be checked and compared with the actual distance that is found below the actual sea/stream bed. In case significant differences contractor should consult the project's owner / designed to take appropriate measures.

The location of cofferdams is better to be described by the pier's center and the structure's centerline. Contractor can then control structure's dimensions, knowing necessary tolerances for footings, drainage, size of struts and walers, etc. Cofferdams should be located by triangulation.

19.2.4. Post Construction / As - Built

General guidelines posed in Section 19.1.4 of this Volume regarding Post Construction / As - Built are also applied here. As-built survey should begin as soon as it becomes feasible - meaning that the actual horizontal and vertical locations of features in the completed structure should be determined as soon as the features are erected.

This is in accordance with the general guideline that final measurements for constructions should be made concurrently with the construction operations.

Original plans must be updated to show any modifications and/or additions that were made during construction phase. In this case, corrections, changes and additions shall be explicitly noted on the final plan sheets.

19.2.5. Accuracy

Construction survey for bridges is challenging in terms of accuracy, which should not surpass 10 mm horizontally and 5 mm vertically, unless otherwise defined by the concerned authority / project's owner (DMAT, 2016).

19.2.6. Note Forms

Just like in road construction, field notes in predefined forms for bridge construction survey should be filled during the construction survey. These notes should contain stakeout listings for all surveyed elements with the actual staked positions noted in them.

19.3. Tunnels

19.3.1. Project Set Out

The pre-construction steps for tunnelling by boring include (Bernhard Maidl, 2012):

- Identification of the geographical extent where construction works will be performed and design a survey control network to cover the entire area.
- Performing reconnaissance survey to identify existing control nearby and establish new stations.
- Setting up a survey control network firmly connecting all stations to the known stations.
- Completing angle and distance field measurements among the stations followed by a Least Squares Adjustment to compute the coordinates of the control stations.

19.3.2. Pre - Construction

The guidelines posed in Section 19.1.2 regarding Pre-Construction works for highways – road projects should be also followed here.

19.3.3. Construction

Several aspects of construction staking mentioned in Section 19.1.3 of this chapter, are also applicable in Tunnel Construction Survey. Additional measures and instructions required, are described in the following paragraphs.

Original and new control network is located on the terrain's surface. Therefore, prior to the initial construction, a secondary control station should be established at the Tunnel Boring Machine (TBM) launching site, and then to the tunnel control station at underground level. Alignment and level control of the excavation tunnel should be done using the established control network and Total Station.

19.3.3.1. TBM procedures

TBM used in tunnel excavation, can never be driven exactly on the intended spatial line. Depending on the TBM type and soil type in the area TBMs either allow rapid corrections or have lethargic reaction behavior.

Surveying in tunnel construction should show TBM's position in relation to the intended line as construction advances, every i.e., half meter. This has been achieved for the first time in real-world conditions with the advance of computer-controlled position fixing (Bernhard Maidl, 2012).

Contemporary surveying instruments allow for permanent monitoring of the TBM's position. This is achieved by receiving a laser light beam at an active receiver system which is permanently installed on the TBM. Using this method, the longitudinal and the rolling of the TBM are computed at the same time (Bernhard Maidl, 2012). Inclinometers measuring roll and pitch angles are also employed in TBMs.

The exact angle of yaw is determined using intelligent sensors as the deflection of the active receiver's system target table longitudinal axis to the laser beam. Differences in horizontal and vertical directions of the point where the laser beam passes through the target table to the intended position together with the roll, pitch, and yaw angles, are used to calculate correct level and position.

After position of the TBM has been determined, the ideal route ahead for the TBM can be calculated. If there are only minor deviations, then the intended route remains the future drive axis. In the case of deviations of several centimeters, software controlling the process calculates the appropriate correction curve, starting from the most recently actually driven curve and leads within the capabilities of the TBM slowly back to the intended curve. Rapid corrections lead to deviations from the ideal axis on the other side (Bernhard Maidl, 2012).

Results are displayed in graphical and numerical form, also in relation with the designed route of the tunnel, as well as the correction curve needed to be transferred back on route. This enables TBM driver to constantly consider machine's reaction while steering. In this way, it is guaranteed that steering operations are in accordance with the current position and that the followed route continuously leads back to the predefined path. Distance between laser and target table is measured and updated with a new tunnel survey for each laser position relocation (Bernhard Maidl, 2012), as follows.

19.3.3.2. Survey control at the tunnel

Tunnel control survey is established with a "working line" meaning the conventional traverse method familiar. Traditionally, in mining tunnels survey control is established on the roof of the tunnel excavation; this stands until recent days. According to (Bernhard Maidl, 2012), at first, the horizontal control station in the form of a bracket is installed at the lower part of tunnel. The station is then plumbed up to the roof of tunnel and the bracket is removed to make way for the construction work. The station is subsequently transferred onto the walkway from the roof station, and it lasts beyond the end of the project.

More sophisticated methods than simple traverses involving multiple measures from one side to several stations on the other side, and least squares adjustment should be employed in tunnel surveys. The zigzag, and double zigzag methods, also proposed in (Bernhard Maidl, 2012), are advised to be used to transfer survey control points in the tunnel. In the double zig-zag method, each traverse leg by itself becomes a quadrilateral. A double zigzag traverse links the quadrilateral.

The double zig-zag method allows the detection of gross error and the orientation of the setup using at least four control stations (Bernhard Maidl, 2012).

19.3.4. Post Construction / As - Built

General guidelines posed in Section 19.1.4 of this Volume regarding Post Construction / As - Built for road projects are also applied here. Original plans must be updated to show any modifications and/or additions that were made during construction phase. In this case, corrections. changes and additions shall be explicitly noted on the final plan sheets.

19.3.5. Accuracy

Tunnel surveys should achieve the following required accuracies (DMAT, 2016):

- Repeated distance measurement spread should be less than 2 mm + 2 ppm.
- Repeated angle measurement spread should not be more than 3”.

It is important to have stable reference points at the beginning of the measurements, which are not affected by the tunnel construction. The accuracy of the TPS employed should be 1 mm + 1 ppm or better for ideal measuring conditions. This accuracy can go down to 3 mm + 1 ppm or worse in difficult conditions, for example, dust or large temperature differences inside the tunnel.

Boring accuracy of 80 - 100 mm in all directions is accepted for tunnels up to 5 km long. Longer tunnels, however, will have larger surveying errors.

19.3.6. Note Forms

Just like in road construction, field notes in predefined forms for tunnel construction survey should be filled during its execution. These notes should contain stakeout listings for all surveyed elements with the actual staked positions noted in them.

19.4. Parking Lots

19.4.1. Project Set Out

Initial step for setting out is the recovery/checking of existing horizontal and vertical control points surrounding the parking lot. If the control is good, contractor/surveyor determines whether additional control is needed for construction purposes and is responsible for staking/establishing it. If not, the field control will have to be reset. Project’s owner and contractor/surveyor should review all control survey data and decide the best method of accomplishing a proper control. After establishing project control, the borderline of the parking lot should be set out by the Consultant/Contractor and be approved by the concerned authority/Client.

19.4.2. Pre - Construction

The guidelines posed in Section 19.1.2 regarding Pre-Construction works for highways – road projects should be also followed here.

19.4.3. Construction

All aspects of construction staking mentioned in Section 19.1.3 of this chapter, are also applicable in parking lot construction survey. Additional measures and instructions required, are described in the following paragraphs.

19.4.3.1. Recording original ground levels of the project

With the end of the initial staking, consultant and contractor should perform a joint survey to determine the original terrain of parking lot corridor to be able to estimate the earthwork quantities after parking construction. Elevations should be gathered at a 25 m grid.

19.4.3.2. Stakes for grading

If the parking lot is built as close to grade as possible with the previous stakes, it will then be necessary to set line stakes and paving stakes at 15 m intervals. These stakes should be of metal of sufficient length to penetrate the grade far enough that the movement of equipment will not cause variations once the grade is established (DMAT, 2016).

19.4.3.3. Stakes for pavement

Prior to the actual pavement staking, field books should be prepared, all computed grades shown on the plans should be verified, whereas any calculated grade for other points should be computed and verified. Given that new pavement is tied into existing pavement, all relative elevations should be checked.

In addition to normal grade stake elevations, it is desirable to include tables of offset from survey line to form stake line when required in the field books. It is a convenience and will save time, if all necessary information from the plans is carefully transferred to the field book.

The Contractor will usually set metal stakes which are not less than 60 cm long and are driven into the ground, normally at 10 m intervals. Location of the string line may vary with each section of the parking lot to be worked. Each section should be evaluated separately to determine the proper location or position of the string line.

Each string line must be set at a constant distance and each string line must also be suspended at a constant height above the plane passing through the lower corners of the proposed slab (DMAT, 2016).

19.4.4. Post Construction / As - Built

General guidelines posed in Section 19.1.4 of this report regarding Post Construction / As - Built are also applied here. As-built survey should begin as soon as it becomes feasible - meaning that the actual horizontal and vertical locations of features in the completed structure should be determined as soon as the features are erected.

This is in accordance with the typical recommendation that final measurements for constructions should be made concurrently with the construction operations.

Original plans must be updated to show any modifications and/or additions that were made during construction phase. In this case, corrections changes and additions shall be explicitly noted on the final plan sheets.

19.4.5. Accuracy

As mentioned above, construction surveys vary on a project-by-project basis, thus required accuracies are generally specified by the Designer/Consultant. Nevertheless, construction survey horizontal and vertical accuracy for parking lots should have an upper limit of 20 mm and 10 mm, respectively (DMAT, 2016).

19.4.6. Note Forms

Just like in road construction, field notes in predefined forms for parking lots construction survey should be filled during its execution. These notes should contain stakeout listings for all surveyed elements with the actual staked positions noted in them.

19.5. Buildings

Construction surveying regarding buildings deals with the exact placement on the ground of the foundations and their footings, piers as well as any other building item is vital for proper building formation. Usual building staking includes the foundation for the excavator to dig the basement or footings, elevations, building corners and appropriate offsets outside the construction area for helping the contractor to place the structure in its correct location without the need of repeating surveying work.

19.5.1. Project Set Out

Initial step for setting out is the recovery/checking of existing horizontal and vertical control points surrounding the building. After setting and checking reference points, the overall boundary of the building site should be demarcated by the contractor with the help of the surveyor and be approved by the project's owner.

19.5.2. Pre - Construction

Carefully studying and checking of the building plans and special provisions shall be performed by the surveyor prior to actual construction/staking work. This can assist in detecting errors as well as to get familiarized with the project. The project engineer should be aware of any significant error or deficiency that will come to the attention of the surveyor regarding the project under examination. Project engineer will then take all the necessary steps to resolve them (IDOT, 2015) Client/project owner should be informed of these and express its opinion.

Pre-construction includes reviewing the plan, preparing notes, performing computations: Field notebooks should be prepared according to uniform practices and conform, as a minimum, to neatness, legibility, clarity, completeness, permanence and accuracy. Field notes recording should follow instructions given in Section 4.4.5.

19.5.3. Construction

Most of the steps described above in Sections 19.1.3.1 to 19.1.3.8 for Road Construction Survey are also applicable to the Building Construction Survey.

19.5.3.1. Recording original ground levels of the project

With the end of the initial staking, consultant and contractor should perform a joint survey to determine the original terrain of the underground basement of the building, so as to be able to estimate the earthwork quantities after construction. Elevations should be gathered at a 25 m grid.

19.5.3.2. Stakes for footings

Precise footing's location and exact location of stumps/posts is a critical step for the correct building construction. Surveyor should check survey information and boundary pegs, ensure that the building is correctly sited on the block and that all local authority requirements are met. Building corners as well as any other building construction element should be staked with temporary pegs using TPS. Direct measurements between marked points with tape/EDM should be performed to check for correct staking.

19.5.3.3. Stakes for column lines

Columns lines should be staked straight and level. On each floor column lines should be set out towards both building directions.

19.5.3.4. Floor setting out

Floor setting out at its correct elevation is supported by string lines. Timber pegs, with timber horizontal rails nailed to the pegs are used for their construction at a determined height which is normally the top of finished floor level. Profile level is determined by the surveyor. Usually, profiles are kept back from building line of approximately 0.9 m when setting out the profiles.

19.5.3.5. Utility setting

Several construction elements are included in the term utilities, such as sewer, water, gas, and oil pipelines, as well as communications and electric power lines. For utilities above the ground simply the location of the line, poles or necessary towers horizontally should be staked. Sometimes pole height for vertical clearance of obstructions has to be also determined.

Underground utilities on the other hand, often require determining both line and grade. Regarding lines under pressure, e.g., water lines, only the line should be staked, given that the only grade requirement is that the prescribed depth of soil cover be maintained. Nevertheless, in the case where lines are place in an area that will be graded downward, or has other, conflicting utilities, staking elevations may be necessary.

Gravity flow lines, like storm sewer lines, require staking for grade to ensure that pipe is correctly installed at the elevation that was designed, and at the provided by the plans slope which is required for gravity flow through that pipe.

19.5.4. Post - Construction / As - Built

General guidelines posed in Section 19.1.4 of this Volume regarding Post Construction / As - Built are also applied here. As-built survey should begin as soon as it becomes feasible and the actual horizontal and vertical locations of features in the completed structure should be determined as soon as these are erected. All the elements of the building construction, including underground and aboveground utilities, together with the total plot are surveyed during and after the construction and put together in the final as-built drawings.

Original plans must be updated to show any modifications and/or additions that were made during construction phase. In this case, corrections. changes and additions shall be explicitly noted on the final plan sheets.

The following list provides some of the information that must be field checked, corrected and added to the original plan sheets (IDOT, 2015):

- Horizontal and vertical control.
- Location, dimensions, and elevations of utilities.
- All underground units (cables, conduits, pipes, etc.).

19.5.5. Accuracy

Construction accuracy regarding footings, columns, floor slabs and utilities, vary on a project-by-project basis; thus, required accuracies are generally specified by the Designer/Consultant. Nevertheless, the construction survey horizontal and vertical accuracy for building projects should have an upper limit of 10 mm and 5 mm, respectively (DMAT, 2016).

19.5.6. Note Forms

Just like in other constructions, field notes in predefined forms for building construction survey should be filled during its execution. These notes should contain stakeout listings for all surveyed elements with the actual staked positions noted in them.

19.6. AMG in Construction Surveys

Automated Machine Guidance (AMG) (Figure 19-1) is a method in constructions that uses data from sources such as 3D engineered models in order to automatically guide the constructing processes and equipment, during procedures such as earth work (grading, milling) and pavement implementation. AMG is defined as the utilization of positioning technologies such as GNSS, robotic TPS, lasers, and sonic systems to automatically guide and adjust construction equipment according to the intended design requirements (White, et al., 2018). The use of

AMG assists tremendously the construction operations if used properly and when necessary: it improves the construction efficiency, safety and quality of the procedures, while it tightens the schedule, reduces the cost and minimizes environmental impacts (DOT, 2013).

Contrary to traditional road construction methods, AMG uses "stringless" control to steer the equipment. Stringless operations don't require string lines, being therefore quicker since they eliminate the need to set and take down the string lines as they move along a project. Furthermore, because there is no need to reroute vehicles around the string line, stringless technology causes fewer traffic interruptions. As a result, there is an improvement in safety for both the construction workers and the general public (IOWA SUDAS, 2018).

While AMG has been used in construction for over two decades, it is a technology that’s evolving and changing along with the technological advancement. However, not all road projects and construction phases are suitable for AMG use. On these projects, different, more traditional methods, may be suitable for application.

Figure 19-1 Example of AMG Used on Construction Site (DOT, 2013)

AMG is mainly used in road corridor construction surveys examined in Section 19.1 of this Volume. Therefore, accuracies and other requirements mentioned there, are also applied in the case of AMG usage.

19.6.1. Concept

AMG delivers automatic/computer aided guidance of construction equipment such as motor graders, dozers etc., in order to follow the planned and decided by the engineering team plan of work. Guidance can be achieved either by direct automatic control of machinery or through visual and/or audible guidance signals to the operator (ODOT, 2021). The guidance provided to the equipment by the AMG refers to the horizontal and vertical control of the equipment used. The abundant advantages of these procedure include (DOT, FHWA, 2013):

- Reduced construction costs as AMG decreases maintenance and fuel cost (environmentally friendly method), agency support costs and at the same time improves machined productivity.
- Reduced schedules and reduced time for survey and staking since it increases equipment productivity and improves logistics.
- Increased quality, increased levels of accuracy and efficiency with greater precision, fewer errors requiring rework since the margin of error is decreased.
- Increased safety since there is reduced need for elimination of string lines for improved worker safety and fewer personnel is exposed to heavy equipment.

It is important to keep in mind that AMG is not ideal for every project; however, it can be applied in a plethora of situations. Types of projects being suitable for AMG employment are (DOT, FHWA, 2013):

- Projects including:
    - Large amounts of construction procedures such as paving, grading, milling etc.
    - A design based on an accurate DTM.
- New alignments and maintenance of already existing alignments.
- An environment that allows the use of GNSS systems for receiving satellite signals or at least an environment that provides enough line of sight for the use of TPS.

On the other hand, projects that are described by the following characteristics may limit the advantages of AMG methods:

- Widening with narrow strip additions.
- Designs that are not based on accurate DTMs.
- Design difficulties that limit the calculation of an accurate DTM.
- Projects that cannot be modeled in a 3D digital environment.
- Projects including structures.
- Lack of training on AMG, of equipment that utilizes the AMG procedures or other restrictions by the project's specification that limit the use of AMG.

AMG is constantly evolving and changing thus, new implementations of AMG methods can be applied in different scenarios and situations.

As a summary, AMG provides a low cost, efficient and accurate method for earthworks (grading, milling) and pavement implementation, based on enhanced data from 3D models and GNSS.

The AMG procedures require the following equipment (Figure 19-2):

- TPS (Robotic or traditional).
- GNSS Base Station, Radio (UHF) modem.
- GNSS Rover.
- GNSS paraphernalia such as antennas, cables, batteries etc.
- Grader, dozier, excavator, or other machinery used for earth work and pavement implementation.

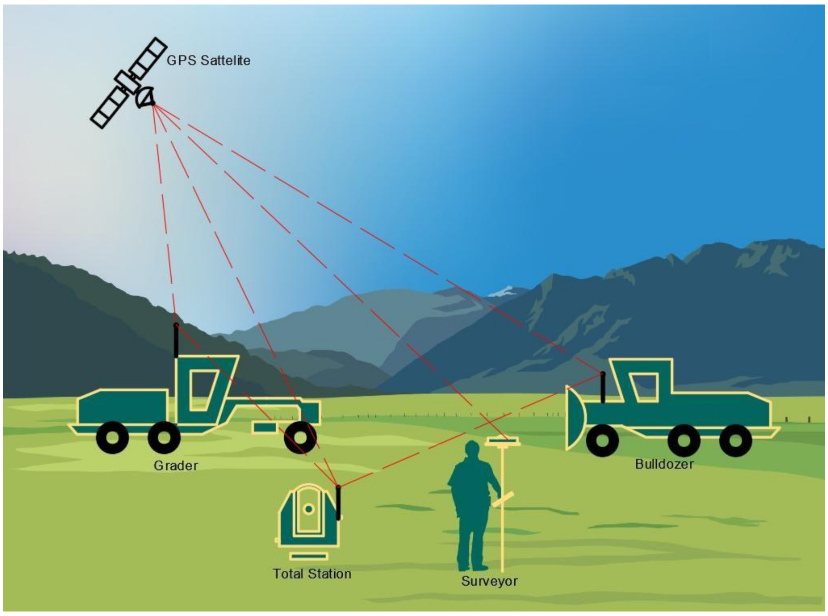


Figure 19-2 Equipment for AMG Methods

19.6.2. Data Preparation and Exchange

In order for the AMG procedures to properly and efficiently work they require the accurate implementation of a 3D model, including surface model or other designed work elements being a digital representation of the line, grade and cross section applicable to the area of the project. The contractor and contractor's team are responsible for maintaining and developing the construction model that will be used by the AMG and the machinery.

As a minimum, the 3D engineered model shall contain the following information (IOWA SUDAS, 2018) :

- Appropriate beginnings and endings of transition for key geometric features.
- Beginnings and endings of vertical and horizontal curves.
- Intersection radius and grading.
- Grading areas such as berms, detention ponds, and ditches.

It is essential for the 3D engineered model provided to the contractor to be continuous, not containing gaps or void spaces within the model boundaries. The contractor is responsible for ensuring that the construction model agrees with the contract plans. If a plan error is discovered, the contractor has to notify the engineer in charge. The Client revises the contract plans and design model, if necessary, to address errors or discrepancies the contractor identifies. The contractor is still responsible for updating the construction model and sending the revised version back to the Client (WDOT, 2021).

The lead engineer should do an error report on the contractor's suggested model. The results of completing spot checks, by projecting known spots created from the plan cross sections onto the suggested model may be included in the error report. The engineer is responsible for maintaining a log of all DTM modifications and their associated dates. The DTM files and the durations for which each was utilized by the project should be included in the archive.

Data exchange between the Client and the Contractor regarding AMG should be in the following data formats (WDOT, 2021), (IOWA SUDAS, 2018):

- Digital Terrain Model (DTM) in 3D DGN or LandXML format.
- Alignments (Horizontal, Vertical and profiles) in LandXML format.
- Cross sections in 2D DGN and pdf format.
- Line strings (2D and/or 3D) and target geometry in DXF and DWG format respectively.
- For other documents such as reports, drawings and/or maps, pdf or docx, are the preferred formats.

LandXML defines a specification for the exchange of cross section data. It includes basic geometry element types, and is readable by several software vendors, mainly Bentley and Autodesk. In addition, LandXML may be consumed by many of the available software packages used in the highway construction industry including AGTEK, Trimble, Carlson, and others. The LandXML format defines data exchange format for basic roadway geometrics including:

- Point data Profiles.
- Curve data Pipe Networks.
- Spiral data Terrain Model Surfaces.
- Alignments (with station equations) Survey Data.
- Cross Sections (surface and design sections).

19.6.3. Procedure

19.6.3.1. Construction staking

Conventional construction staking requirements can be reduced for machine guidance to increase cost savings. Reducing the requirements for construction staking also improves safety for surveyors traversing the construction site while near to large equipment. Construction staking, although limited, is still required at reduced intervals to ensure that the 3D engineered model and construction equipment is giving the correct layout and elevations. This provides another layer of quality control for a more accurate finished product (IOWA SUDAS, 2018).

19.6.3.2. Survey control for AMG grading

For proper use of AMG technology, accurate survey control is a necessity. Therefore, a control network should be recovered, checked and densified on site prior to any machinery control use, during the project set out phase of the construction survey. The AMG system needs to be able to reference coordinate’s location by using a GNSS base station, that transmits corrections to the receiver that is attached to the associated AMG equipment. The survey control set up should contain at least one monument that remains undisturbed throughout construction. The monument(s) are positioned in order to bracket the area of the construction and use them as check point(s) throughout the process of the project.

19.6.3.3. Survey control for stringless operations

String lines have traditionally been used to control paving machine elevation and steering on the grade. On the other hand, stringless paving (sometimes referred to as “3D paving”) is the process of constructing a pavement using non-contact, electronic guidance systems to guide the paver along the grade without the aid of string lines. Stringless paving and milling machines typically use total stations or laser augmented GNSS for machine control purposes because paving and milling operations require more accurate and tighter tolerances in comparison to the grading operations. As with machine control grading, stringless guidance systems need to able to identify its exact location (X, Y, Z coordinate).

Survey and machine control for stringless operations are different from the procedures used in survey control for AMG grading. For total stations a survey establishing new control points has to take place. Generally, control points are set along the project corridor. Control points for stringless paving should be established from accurate field surveying and tied to known benchmarks. The control points shall be established in locations where they do not intervene with the operations performed, they won't be disturbed by the public, are securing clear line of sight, and at least three control points can be always seen simultaneously by the surveying equipment used.

During stringless paving, the location and the elevation of the finished slab should be verified against grade check hubs for the first 30 meters of each run and at specific locations of interest, such as intakes and through intersections where grades may be flat. If compliance with the design elevations is secured then these requirements may be waived (White, et al., 2018).

After each modification to the paving machine, calibration for the paving equipment is required and strongly advised as per the manufacturer's recommendations, unless other standards have been set by the Client.

19.6.4. Guidelines

The following guidelines should be considered when dealing with AMG in road construction (White, et al., 2018).

- Contractor may use any type of AMG system(s) that result in compliance with the contract, documents, and applicable Standard Specifications.
- The contract plans should indicate areas of the project where AMG systems may be used for roadway construction. In these areas the contracting authority should provide DTM of the roadway embankment construction. Conventional survey and construction techniques will be used in all other areas unless the contractor chooses to build the required surface model to facilitate AMG grading for those areas at no additional cost to the contracting authority.
- Final surface grades shall not be constructed with GNSS usage.
- The contracting authority may require the contractor to revert to conventional subgrade staking methods for all or part of the work at any point during construction if, in the engineer's opinion, the GPS machine guidance is producing unacceptable results.
- The contractor shall convert the electronic data provided by the contracting authority into the format required by their system.
- If the contractor choses to use furnished DTM data, he shall be fully responsible for all cost, liability, accuracy and delays, and he shall release contracting authority and its employees from all liability for the accuracy of the data and its conformance to the contract.
- The contracting authority does not guarantee the electronic data accuracy or completeness, or that the data systems used by the contracting authority are directly compatible with the systems used by the contractor.

- The contractor may use any type of automated machine control (AMG) systems that achieves compliance with the contract documents and applicable standard specifications.
- The contracting authority shall provide project alignment, and coordinate system information as well as the initial horizontal and vertical control network.
- The contracting authority should provide electronic files for areas marked for AMG usage in the following formats (alternatively).
    - Bentley's MicroStation suite of road design software.
    - DTM of the existing and proposed design surface, ASCII format - Machine Control Surface Model Files, ASCII Format - Alignment Data Files.
    - CADD files, Machine Control Surface Model Files (ASCII/LandXML), Alignment Data Files (ASCII/LandXML).
- The contracting authority shall approve or certify any changes to the DTM used by AMG prior to its usage for grading operations.
- Prior to any usage of AMG equipment, the contractor shall demonstrate to the contracting authority its capabilities and tolerances on a special test session. In the case where the equipment fails to meet the tolerance standards, the contractor shall construct the project using conventional survey and construction methods.
- Contractor shall validate all control points provided by the contracting authority, prior to their usage. He will also densify control network establishing secondary control points at locations along the length of the project and outside the project limits as required by the AMG system utilized.
- Controls points and conventional grade stakes shall be provided by the contractor at critical points such as, PC's, PT's and other critical points required for the construction of drainage and roadway structures.
- The site calibration shall be checked daily at control points not used in the calibration.
- All reference points and monuments established by the contracting authority outside the construction limits shall be preserved and re-established if necessary.
- The contractor shall set grade stakes and such that the contracting authority can check the accuracy of the construction.
- In the case where the contractor uses AMG for fine grading and placement of base or other roadway materials, the AMG system shall use a laser or robotic total station.
- Contractor shall check and recalibrate, if necessary, their AMG system at the beginning of each work day to ensure compliance with contract documents.

20. Road Maintenance and BIM

20.1. Road Maintenance and Surveying

Survey works are applied during road operation and maintenance in a variety of circumstances: monitoring of existing constructions and roadway's surface, pavement maintenance, road utility maintenance, new constructions nearby or onto the road surface for utility works, minor road improvements etc. Given that the efficient execution of these works, requires the design and construction control network, it should be properly maintained after road construction is completed.

20.2. Project Control

During utility works on or nearby the road corridor and into the RoW, the following situations may occur, affecting survey works:

- Survey monument disturbance or destruction due to utility work.
- Preventing future survey operation interference by a utility facility.

In such situations, the concerned authority should be provided with a sufficient notice when a survey monument must be relocated and there is no other viable location for placing a utility facility within its RoW.

Survey monuments can be disturbed by utility excavation, either through soil collapsing on them or pushing dense soil. In sandy or loose soils, or when excavating deeper than 1.5 m, the conditions are worse. Moreover, monuments may be disturbed by improper backfilling causing settlements. When making deep excavations, the respective removed soil will cover larger areas. Therefore, in order to avoid covering it with excavated dirt or disturbing it with backfilling, monuments have to be visible to the excavator. Hitting, vibrating, or depositing equipment that runs over, on or near a monument can also disturb it (and it will probably do).

Regarding new utility facilities, proper placement aboveground should consider that several infrastructures such as overhead lines, poles and guy wires may interfere with signals from the GNSS satellites. Aboveground utility facilities such as poles, pedestals, cabinets, guy anchors, etc., are typically placed close to the RoW line to keep them out of the clear zone, where survey monuments are also located. Therefore, such utility facilities have to be placed away from survey monuments so as to minimize the disturbance of possible survey works in the nearby monuments, and to prevent them from being moved or destroyed (WSDOT, 2021).

During placing a utility, the following shall be done to avoid disturbing monuments:

- Monuments and witness/guard posts shall be surrounded with orange safety fence to make the monument more visible for protection with a 2 m radius.
- A lath painted pink and/or with pink survey flagging shall be placed nearby to visually indicate the monument location if a monument does not have a witness post.
- Soil or debris should not be placed on a monument. If a soil pile spills onto a monument, the material should be removed by hand or a soft brush.

20.3.BIM Procedures and Technologies

Another major survey work that may take place during roadway's operation originates from the need for updating its as-built information towards the employment of a BIM. BIM is an Information model that represents structural elements using 3D digital technology while accommodating them with a data source that contains various related information about those elements. Additionally, the 3D modeled elements carry not only geospatial information, rather than contain metadata information that describes those element's attributes. BIM surveys should take place with MMS which are ideal for this kind of use.

In recent years, the concept of the so-called Digital Twins has also evolved. A digital twin is a digital representation of a planned or actual real-world physical infrastructure system that serves as the effectively identical digital counterpart of it for practical purposes, such as simulation, integration, testing, monitoring, and maintenance. Digital twins combine the power of physics-based simulation and real-time sensors; therefore, rather than looking like the building, a digital twin behaves like it, responding to operational conditions etc. In this sense, a digital twin is much more focused upon building/structure performance than a typical BIM model

BIM aims at the smoother collaboration and communication amongst all the authorities/agencies/departments etc., included during the development of road and road infrastructures related projects. Improvement in productivity, business processes, transparency, reduction in errors among the parties included can be succeeded by the use of BIM. BIM offers object oriented parametric modelling, which means that when a change is applied in an element the others elements adjacent to it are automatically adjusted, in order to maintain any prior established relationships among the components effected (Azhar, et al., 2012).

BIM has a substantial impact on the entire processes conducted during highway projects. BIM is as intelligent 3D model-based strategy and at its crux consists of 3D models/components that are represented digitally by objects that contain parametric features and metadata information. All data are collected in a single source of information that is accessible to all parties and can be updated easily, e.g., a modern relational database. Creating a single source of information establishes the best possible cooperation among teams participating in the development and helps to minimize errors and mistakes (UN, 2021).

BIM employment ensures an uninterrupted flow of data, processes and tools for communication; during a BIM process, a model is created and used through all stages of development. This model can be constantly refined and adjusted by all members, according to the specifications and design changes decided, simultaneously it can be reviewed in order to ensure that the best options have been made before the physical implementation of the project (Azhar, et al., 2012).

20.3.1. Tools and Technologies

BIM design tools include aspects from 2D CADD to 3D and object-oriented modelling. BIM is essentially based on parametric modelling technologies. Parametric modelling offers the most enhanced level of information modelling of an infrastructure component and is more efficient in comparison to other CADD technologies (UN, 2021).

Object based parametric modelling utilizes features called parameters, in order to exhibit the properties of each object along their relationships with other objects. The properties can contain, spatial information in addition to non-spatial features such as manufacturer information, vendor, materials used, code requirements and any other information related to how this object is used (UN, 2021). A draining pipeline can be used as a representative example where several geometric data, hydraulic characteristics, materials, etc., are presented for each element. The evolution of the 3D models constructed, and their object's properties, defined can be categorized into different phases. Early phases include the more basic description of the model and its objects and as advancing through the phases, the model and the objects get more accurate with more information portraying them. This process in known as Level of Development (LOD). LOD uses codes, in order to describe the condition and the phase of each object. The codes are part of BIM standards and their definition as established by the BIM Forum's 2013 Draft LOD Specification (FIU, 2014):

- LOD 100: Conceptual, basic form of object, no geometric representations.
- LOD 200: Generic Placeholders, graphical representation of object.
- LOD 300: Specific Assemblies, accurate geometry, size, and graphical representation of an object that can be measured without referring to non modelled information.
- LOD 350: Hybrid of Specific and Detailed Assemblies, accurate (geometry, size) graphical representation of an object within the model, descriptive information may be included.
- LOD 400: Detailed Assemblies, the quantity, size, shape, location, and orientation of the element as designed can be measured directly from the model without referring to non-modelled information such as notes or dimension callouts.
- LOD 500: As - built, The Model Element is a field-verified representation in terms of size, shape, location, quantity, and orientation. Non-graphic information may also be attached to the Model Elements."

BIM is not defined by a single process or software; on the contrary it is a multi-layered technology that requires the coordination of different cross functional processes and software. The tools required for the successful completion using BIM can be broken down to different categories based on the use of each tool and the phase of process in which they are used (preliminary, construction) (UN, 2021).

- Authoring tools that are used to create the actual model.
- Analysis tools are used to analyze and predict model's behavior, for highway projects specifically, the most used tools include traffic, visibility, noise, and lighting analysis.
- Validation tools to ensure the accuracy and precision of the model and that complies with the requirements and standards.

20.3.2. BIM in Road Design

BIM tools are mainly used to determine the optimal location for the construction of the infrastructures. Optimal location selection is a dynamic procedure, based on the advantages and disadvantages, for every possible location, which can be achieved by employing BIM processes. Since BIM offers an enhanced visualization of the model and their objects, concerned authorities can understand beforehand the potential economic and environmental effects of road design. In road projects, the visualization of the BIM model assists in the inspection of the model form various angles: the inspection for instance can occur from the driver's perspective. These reviews assist to the identification of possible black spots helping to suggest possible improvements (UN, 2021).

The 3D models produced by BIM procedures include representations of the civil and structural design, with other designs such as services, traffic, lightning, etc. Authoring design tools are responsible for the connection of elements/objects from 3D models with a database containing descriptive information such as schedules, materials, costs etc. BIM provides tools such as, traffic simulation, cut/fill quantities, site check, flood, water shed analyses etc. that can operate as a review mechanism of the 3d model designed. These analyses can also operate as simulation tests in order to examine the 3D model's behavior in specific scenarios (Eastman, et al., 2008).

Code validation is used in BIM software in order to compare the model parameters with project's codes, standards and requirements. Generally, these codes represent vital regulatory obligations for road design and safety. BIM tools can assist with cross referencing established standards with the design constrains, such as speed limits, elements of road alignment etc. (UN, 2021).

Furthermore, BIM offers the ability to automatically calculate the extracted quantities, from the model and instantaneously estimate the cost of the project. In addition, this process of utilizing BIM can help to compare the costs of various designs to make changes at the early design stages, helping to avoid budget overruns. BIM tools for calculating quantities, are more accurate than the calculations from traditional methods which are basically estimations using 2D drawings (UN, 2021).

When 3D model is reviewed and verified successfully, then 4D and 5D analyses can take place that offer a dynamic view of the concept of the project as far as it concerns the aspects of duration and cost. 4th dimension is used to present visualizations showing the development of the project throughout its duration, and can assist to the better scheduling and planning, of activities on site while securing no collision between such activities, occurs. 5th dimension representations are created when adding the parameter of cost to the parameters of time, location (X, Y) and elevation (Eastman, et al., 2008).

20.3.3. BIM During Construction Surveys

The BIM process outputted 3D model serves as the main input for AMG construction surveys (Section 19.6). That way the accurate progress and implementation of the project is ensured. Generally, most machinery on site are controlled and manipulated wirelessly by computers. GNSS receivers are attached to road construction machines that control their behavior and routes followed. Once the model has been created it is referenced to the site's GNSS coordinates and loaded onto the site equipment computers (UN, 2021).

Moreover, BIM technology contributes during construction surveys, ensuring an efficient and fluent workflow while it can assist to the coordination of site activities, teams, minimizing conflicts and eliminating miscommunications. BIM assists to the design of site's layout scheme so that it can be automatically generated and dynamically adjust. Site offices, storage areas, entrances, temporary roads other facilities, etc. should be included in the 3D BIM model with the appropriate information.

Additionally, BIM can be used through mobile devices in real time on the site. Thus, BIM guarantees the accurate implementation of the model while, reviewing and tracking in real time the construction's progress and eliminating possible incidents of omissions and human errors.

20.3.4. BIM for As - Built Surveys

As-built survey output can be expressed into a BIM model, rather than simple drawings; such as-built surveys can be performed throughout the road construction process, or during its operation. As-build surveys output into BIM are conducted with stationary TLS and Mobile Mapping systems, outputting massive point clouds. Then point cloud data can be imported into a BIM authoring software used for creating constructible model content.

The process is realized by 5 main steps, i.e., point cloud acquisition, point cloud processing, modeling, visualizing and final delivering. Point cloud acquisition and processing are performed using Mobile Mapping systems or simple TLS, in the case of limited site size, using the procedures demonstrated in Chapters 14 and 15 of this code. Modeling and visualizing process should include the following:

- Terrain extraction from point cloud.
- Automatic feature extraction from point cloud.
- Manual feature extraction from point cloud.
- Exporting point and features.
- Exporting road markings.
- Viewing point cloud along with extracted points features and road markings.
- Component mapping road corridor materials.
- Review corridor materials.
- Review line marking.
- Completed model preview with and without point cloud.

Conclusively, as - built and terrain capture can be automated by embracing 3D Laser Scanning Technologies whereas importing, registering, indexing & cropping scan data can be achieved with modern software. Then point cloud feature extraction can be utilized for accurate corridor targeting & modelling, while road line markings and paint work can assist to produce realistic visualizations of the final BIM model.

21. Exception Procedure

In all survey works for road projects specified in this code, there are certain values and procedures described that have been determined to be paramount.

The content of this code defines currently accepted and generally established rules, appropriate surveying methods, and equipment for accurately conducting mapping activities for highway projects. Methodologies, procedures, design values, and acceptable maximum error values expressed throughout the code are binding, but exceptions in specific cases are allowed if they are properly supported and approved as follows.

When it is determined that it is not practical for these values and procedures to be met, documented justifications shall be submitted, and approval must be obtained for inclusion in the survey results. Purpose of next sections is to specify the exception justification requirements, and the procedures required to obtain needed approvals.

21.1. Elements Requiring Exceptions

Five survey elements included in this code may require exceptions:

- **Chosen Methodologies** by scale and project phase, according to what is described in Table 3-1 and Table 3-4.
- **Survey Procedures** applied during the actual field surveying and during the office processing steps taken afterwards. For example, survey procedures are the multiple pass (Section 15.2.3.1) and data processing (Section 15.3) requirements in Mobile Mapping.
- **Supporting Procedures**, are procedures regarding instruments etc., applied prior to field surveying. For example, procedures regarding calibration, and instrument care in Mobile Mapping (Sections 15.2.1 and 15.2.2) are supporting procedures.
- **Survey Design Values**, are acceptable values of parameters chosen regarding survey design, prior to performing the actual surveying. For example, values of Table 3-5 and Table 3-6 are survey design values.
- **Maximum Acceptable Error Values**, are the permittable residuals for each surveying process, selected according to the chosen methodology, scale, and order, as described throughout the code. For example, values of Table 2-1, Map Scales and Horizontal Accuracies are in this category.

21.2. Process

Pre-survey

A justification letter requesting approval for the Chosen Methodologies, Survey Design Values and Maximum Acceptable Error Values is prepared by the survey engineer of the competent authority and submitted to MoTLS, prior to employing the survey consultant.

Survey

During the execution of the actual survey by the consultant, the need for exceptions in surveying methodologies and survey design values should be minimized with a thorough analysis during the scoping or pre-design stage. It is, however, not uncommon in the refinement of survey design in the actual survey procedure, after obtaining more precise design data, that the need for exceptions in surveying methodologies and design values becomes evident.

Prior to the actual survey field work beginning, exceptions may also be justified for survey and supporting procedures. It is important that the surveying engineer address these changes from the scoping document early and in an expedient manner. By addressing the potential exceptions in the early surveying stage, impacts on schedule and project cost can be minimized.

All the controlling elements requiring exceptions (methodologies, survey and supporting procedures, design values, acceptable errors) should be refined and identified prior to finishing the survey on-site. Only rarely and with strong justification exceptions will be given to requests issued at this stage. Exceptions will not be granted based upon a lack of adequate time to make changes to meet the project schedule and project requirements.

Exception requests with appropriate justification will be sent from the survey engineer in charge to the competent authority for review and acceptance. Competent authority may also submit requests to MoTLS for review and approval. The survey engineer shall keep all exception requests and approvals in the project file and send copies to any concerned authority.

21.3. Evaluation and Justification for Exception Procedure

Requests for exceptions must be accompanied by appropriate justification. Approval of an exception requires compelling reasons to justify why the established standard cannot or should not be used.

Consideration must be given for the effects of the variance from the standard on the accuracy of the final survey outcome, on the safety and operation of the facility and the compatibility with adjacent sections of highway. Consideration must also be given to the functional classification of the highway, the type of project (e.g., new or reconstruct).

The cost of obtaining current applicable standards should be weighed with any resultant impacts upon scenic, historic, and other environmental features. Future planned improvements to the roadway or corridor must be considered.

Issues to be considered in any analysis should include:

- What is the degree to which the standard is being reduced?
- What impacts, if any, will the exception have on other standards?
- Does a reduction in the standard significantly impact the accuracy of the survey outcome, safety in the specific area, or the overall project?
- Are there any other additional features that would mitigate the impacts of the deviation from standard?

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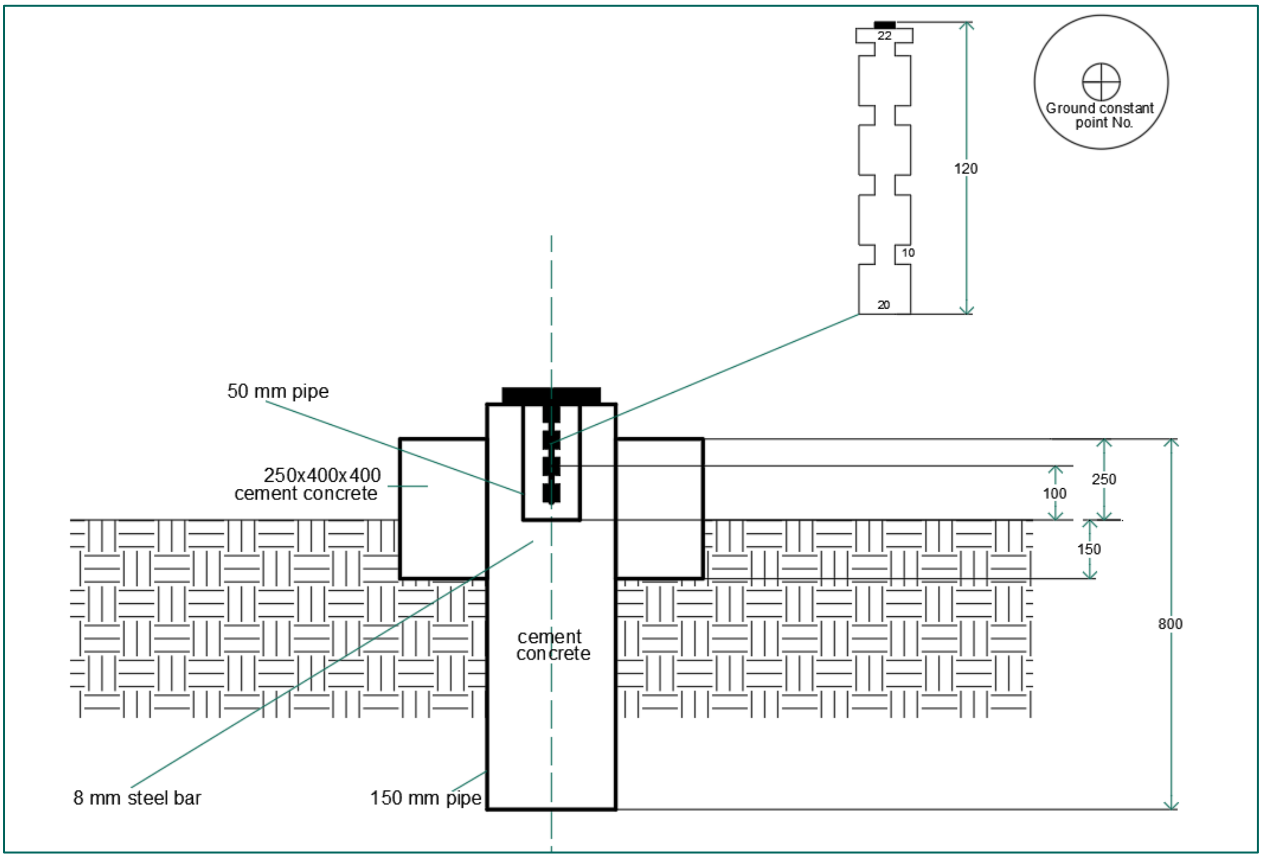
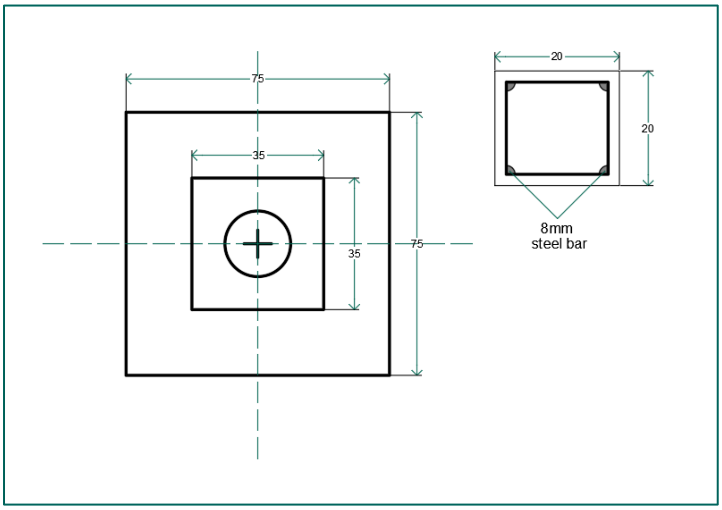
Appendix A - Monuments and Feature Codes

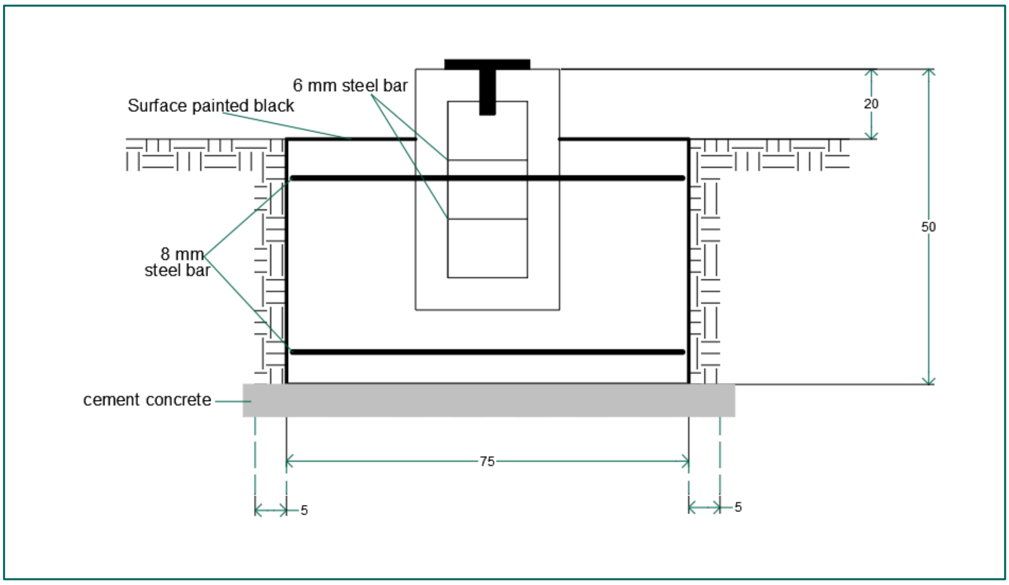
A.1. Establishing Monuments (MoMRAH, 2005)



Dimensions in millimeters

Figure A-1 Monuments of First and Second Order (MoMRAH, 2005)





Dimensions in millimeters

Figure A-2 Monuments of Third Order (MoMRAH, 2005)

A.2. Feature Codes for Topographic Maps, Scale 1:1,000

Table A-1 Feature Codes for Topographic Maps, Scale 1:1,000 (MoMRAH, 2005)

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
1	Airport Area Outline	1001	1	3	0	5	----	0	-	√
2	Built Up Area Outline	1002	2	3	1	0	----	0	-	√
3	Built Up Area Pattern	1003	2	3	0	0	----	0	-	√
4	Built Up Area Polygon	1004	2	9	0	0	----	2	-	√
5	Cemetery Outline	1005	3	5	0	5	----	0	-	√
6	Cemetery Pattern	1006	3	5	1	0	1006	0	-	√
7	Cemetery Polygon	1007	3	5	0	0	----	2	-	√
8	Landing Strip Outline	1008	1	9	0	5	----	0	-	√
9	Landing Strip Symbol	1009	1	9	1	0	1009	0	-	√
10	Misc. Area Outline	1010	5	4	0	5	----	0	-	√
11	Park Outline	1011	3	2	0	5	----	0	-	√
12	Park Symbol	1012	3	2	1	0	1012	0	-	√
13	Park Polygon	1013	3	2	0	0	----	2	-	√
14	Pit Outline	1014	4	5	0	5	---	0	-	√
15	Pit Symbol	1015	4	5	1	0	1015	0	-	√
16	Quarry Outline	1016	4	6	0	5	----	0	-	√
17	Quarry Symbol	1017	4	6	1	0	1017	0	-	√
18	Recreation Area Outline	1018	3	1	0	5	----	0	-	√
19	Recreation Area Symbol	1019	3	1	1	0	1019	0	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
20	Refuse Dump Outline	1021	3	6	0	5	----	0	-	√
21	Refuse Dump Symbol	1022	3	6	1	0	1022	0	-	√
22	Refuse Dump Polygon	1023	3	6	0	0	----	2	-	√
23	Bench Mark Annotation	1101	34	3	1	0	----	0	1	√
24	Contour-Index Annotation	1102	43	6	2	0	----	0	2	√
25	Contour-Intermediate Annotation	1103	44	5	0	0	----	0	2	√
26	Control Point Annotation	1104	34	3	1	0	----	0	1	√
27	General Label	1105	57	0	1	0	----	0	3,76	√
28	Government Building Annotation	1106	55	6	1	0	----	0	3,76	√
29	Graticule Numbers	1107	56	3	1	0	----	0	6,76	√
30	Hydrographic Annotation	1108	57	1	1	0	----	0	4,76	√
31	Landmark Area Annotation	1109	57	3	1	0	----	0	3,76	√
32	Misc. Feature Annotation	1110	57	6	1	0	----	0	3,76	√
33	Place Name Annotation	1111	58	3	2	0	----	0	3,76	√
34	Section Name Annotation	1112	59	2	4	0	----	0	3,76	√
35	Spot Elevation Annotation	1113	48	6	1	0	----	0	2	√
36	Street Name Annotation	1114	60	3	1	0	----	0	3,76	√
37	Title Annotation	1115	56	0	3	0	----	0	5,76	√
38	Utility Annotation	1116	57	5	1	0	----	0	3,76	√
39	Utm Grid Annotation	1117	56	0	1	0	----	0	2	√
40	Spot Elev. Rooftop Annotation	1118	48	1	1	0	----	0	2	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
41	Graticule Lines	1201	56	3	1	0	----	0	-	√
42	Graticule Ticks	1202	56	3	1	0	1202	0	-	√
43	Neat Line	1203	56	8	0	0	----	0	-	√
44	Registration Marks	1204	56	0	0	0	1204	0	-	√
45	Surround Line	1205	56	3	1	0	----	0	-	√
46	Trim Line	1206	56	2	1	0	----	0	-	√
47	Utm Grid Lines	1207	56	4	0	0	----	0	2	√
48	Utm Grid Ticks	1208	56	4	1	0	1208	0	-	√
49	Airport Building	2001	1	3	3	0	----	0	-	√
50	Airport Symbol	2002	1	3	1	0	2002	0	-	√
51	Building-General-Outline	2003	6	0	2	0	----	0	-	√
52	Building-General Pattern	2004	2	0	2	0	----	0	-	√
53	Bldng-General-Symbol-Rect	2005	7	0	1	0	2005	0	-	√
54	Bldng-General-Symbol-Square	2006	7	0	1	0	2006	0	-	√
55	Building-Government	2007	8	6	3	0	----	0	-	√
56	Building-Government-Symbol	2008	8	6	1	0	2008	0	-	√
57	Building-Ruin	2009	6	0	1	1	----	0	-	√
58	Building-Under Construction	2010	6	0	1	2	----	0	-	√
59	Castle Building	2011	9	4	3	0	----	0	-	√
60	Castle Symbol	2012	9	4	1	0	2012	0	-	√
61	Filling Station Building	2013	10	2	3	0	----	0	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
62	Filling Station Symbol	2014	10	2	1	0	2014	0	-	√
63	Fire Station Building	2015	8	3	3	0	----	0	-	√
64	Fire Station Symbol	2016	8	3	1	0	2016	0	-	√
65	Generator/Sub-Station Bldng	2017	11	5	2	0	----	0	-	√
66	Generator/Sub-Station Symbol	2018	11	5	1	0	2018	0	-	√
67	Lighthouse Building	2019	8	5	3	0	----	0	-	√
68	Lighthouse Symbol	2020	8	5	1	0	2020	0	-	√
69	Medical Facility Building	2021	8	2	3	0	----	0	-	√
70	Medical Facility Symbol	2022	8	2	1	0	2022	0	-	√
71	Microwave Relay Station Bldng	2023	11	3	2	0	----	0	-	√
72	Microwave Relay Station Symbl	2024	11	3	1	0	2024	0	-	√
73	Mosque Building	2025	12	2	3	0	----	0	-	√
74	Mosque Symbol	2026	12	2	1	0	2026	0	-	√
75	Mosque Building (Friday)	2027	12	3	3	0	----	0	-	√
76	Mosque Symbol (Friday)	2028	12	3	1	0	2028	0	-	√
77	Municipality Building	2029	9	3	3	0	----	0	-	√
78	Municipality Symbol	2030	9	3	1	0	2030	0	-	√
79	Police Station Building	2031	8	0	3	0	----	0	-	√
80	Police Station Symbol	2032	8	0	1	0	2032	0	-	√
81	Post Office Building	2033	8	4	3	0	----	0	-	√
82	Post Office Symbol	2034	8	4	1	0	2034	0	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
83	Public Telephone Office Bldng	2035	11	6	3	0	----	0	-	√
84	Public Telephone Office Symbol	2036	11	6	1	0	2036	0	-	√
85	Public Toilet Building	2037	10	1	2	0	----	0	-	√
86	Public Toilet Symbol	2038	10	1	1	0	2038	0	-	√
87	Pumping Station-Oil Building	2039	13	3	3	0	----	0	-	√
88	Pumpiaw Station-Oil Symbol	2040	13	3	1	0	2040	0	-	√
89	Pumping Station-Water Bldng	2041	13	1	3	0	----	0	-	√
90	Pumping Station-Water Symbl	2042	13	1	1	0	2042	0	-	√
91	Roof Level Change	2043	14	6	0	0	----	0	-	√
92	School/College/University Bldng	2044	8	1	3	0	----	0	-	√
93	School/College/University Symbl	2045	8	1	1	0	2045	0	-	√
94	Fence-Brick To Scale	3001	15	1	1	0	----	0	-	√
95	Fence-Brick Symbolized	3002	15	1	1	4	----	0	-	√
96	Fence-Iron And Brick	3003	15	2	1	8	----	0	-	√
97	Fence-Wire	3004	15	4	0	4	----	0	-	√
98	Gate	3005	15	6	1	1	----	0	-	√
99	Land Boundary	3006	16	7	1	7	----	0	-	√
100	Masonry-Sloping Outline	3007	17	1	1	0	----	0	-	√
101	Masonry-Sloping Pattern	3008	17	1	0	3	----	0	-	√
102	Masonry-Vertical	3009	17	1	1	6	----	0	-	√
103	Miscellaneous Structure	3010	18	6	1	0	----	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
104	Miscellaneous Feature	3011	18	6	0	3	----	0	-	√
105	Stationary Crane Outline	3012	18	3	1	0	----	0	-	√
106	Stationary Crane Symbol	3013	18	3	1	0	3013	0	-	√
107	Steps Outline	3014	17	1	1	2	----	0	-	√
108	Steps Interior Pattern	3015	17	10	0	0	----	0	-	√
109	Swimming Pool Outline	3016	18	1	1	0	----	0	-	√
110	Swimming Pool Symbol	3017	18	1	1	0	3017	0	-	√
111	Airport Tarmac Outline	4001	1	3	1	0	----	0	-	√
112	Bridge	4002	19	9	2	0	----	0	-	√
113	Car Park-Paved Outline	4003	20	3	1	0	----	0	-	√
114	Car Park-Unpaved Outline	4004	20	3	1	3	----	0	-	√
115	Car Park Symbol	4005	20	3	1	0	4005	0	-	√
116	Culvert	4006	19	3	1	0	----	0	-	√
117	Cutting	4007	19	9	1	0	----	0	-	√
118	Cutting Without Line	4008	19	9	1	1	----	0	-	√
119	Embankment	4009	19	11	1	0	----	0	-	√
120	Embankment without Line	4010	19	11	1	1	----	0	-	√
121	Ferry Route	4011	21	3	0	5	----	0	-	√
122	Ferry Symbol	4012	21	3	1	0	4012	0	-	√
123	Footbridge	4013	19	9	2	0	----	0	-	√
124	Footpath	4014	22	3	1	5	----	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
125	Railway Barrier	4015	21	9	1	0	----	0	-	√
126	Railway Line Single	4016	21	10	1	4	----	0	-	√
127	Railway Line Double	4017	21	3	1	6	----	0	-	√
128	Road-Shoulder	4018	23	9	0	0	----	0	-	√
129	Road-Paved Edge	4019	23	3	1	0	----	0	-	√
130	Road-Paved Edge Clx	4020	23	9	1	0	----	0	-	√
131	Road-Paved Centerline -- Clx	4021	23	9	0	3	----	2	-	√
132	Road Paved Centerline -- Sym	4022	23	9	1	0	----	0	-	√
133	Road Paved >15m-- Ncr Clx	4023	23	1	3	5	----	2	-	√
134	Road Paved >15m-- Scr Clx	4024	23	1	3	2	----	2	-	√
135	Road Paved >15m-- Dcr Clx	4025	23	1	3	3	----	2	-	√
136	Road Paved >15m-- Ncr Sym	4026	23	1	1	5	----	0	-	√
137	Road Paved >15m-- Scr Sym	4027	23	1	1	2	----	0	-	√
138	Road Paved >15m-- Dcr Sym	4028	23	1	1	3	----	0	-	√
139	Road Paved 10-15m -- Ncr Clx	4029	23	2	3	5	----	2	-	√
140	Road Paved 10-15m -- Scr Clx	4030	23	2	3	2	----	2	-	√
141	Road Paved 10-15m -- Dcr Clx	4031	23	2	3	3	----	2	-	√
142	Road Paved 10-15m -- Ncr Sym	4032	23	2	1	5	----	0	-	√
143	Road Paved 10-15m -- Scr Sym	4033	23	2	1	2	----	0	-	√
144	Road Paved 10-15m -- Dcr Sym	4034	23	2	1	3	----	0	-	√
145	Road Paved7-10m -- Ncr Clx	4035	23	4	2	5	----	2	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
146	Road Paved7-10m -- Scr Clx	4036	23	4	2	2	----	2	-	√
147	Road Paved7-10m -- Dcr Clx	4037	23	4	2	3	----	2	-	√
148	Road Paved7-10m -- Ncr Sym	4038	23	4	1	5	----	0	-	√
149	Road Paved7-10m -- Scr Sym	4039	23	4	1	2	----	0	-	√
150	Road Paved7-10m -- Dcr Sym	4040	23	4	1	3	----	0	-	√
151	Road Paved5- 7m -- Ncr Clx	4041	23	5	2	5	----	2	-	√
152	Road Paved5- 7m -- Scr Clx	4042	23	5	2	2	----	2	-	√
153	Road Paved5- 7m -- Dcr Clx	4043	23	5	2	3	----	2	-	√
154	Road Paved5- 7m -- Ncr Sym	4044	23	5	1	5	----	0	-	√
155	Road Paved5- 7m -- Scr Sym	4045	23	5	1	2	----	0	-	√
156	Road Paved5- 7m -- Dcr Sym	4046	23	5	1	3	----	0	-	√
157	Road Paved3- 5m -- Ncr Clx	4047	23	6	2	5	----	2	-	√
158	Road Paved3- 5m -- Scr Clx	4048	23	6	2	2	----	2	-	√
159	Road Paved3- 5m -- Dcr Clx	4049	23	6	2	3	----	2	-	√
160	Road Paved3- 5m -- Ncr Sym	4050	23	6	1	5	----	0	-	√
161	Road Paved3- 5m -- Scr Sym	4051	23	6	1	2	----	0	-	√
162	Road Paved3- 5m -- Dcr Sym	4052	23	6	1	3	----	0	-	√
163	Road Paved Median -- Scr Sym	4053	23	3	1	6	----	0	-	√
164	Road Paved >10m-- Clx	4054	23	9	3	3	----	2	-	√
165	Road Paved5-10m -- Clx	4055	23	9	2	2	----	2	-	√
166	Road Paved <5m-- Clx	4056	23	9	1	5	----	2	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
167	Road Paved >10m-- Sym	4057	23	3	1	3	----	0	-	√
168	ROAD Pavaw5-10M -- Sym	4058	23	3	1	2	----	0	-	√
169	Road Paved <5m-- Sym	4059	23	3	1	5	----	0	-	√
170	Road-Unpaved Edge	4060	24	1	0	3	----	0	-	√
171	Road-Unpaved Edge-Clx	4061	24	9	0	2	----	0	-	√
172	Road-Unpaved Cntrln-Clx	4062	24	1	2	1	----	2	-	√
173	Road-Unpaved Cntrln- Sym	4063	24	1	0	5	----	0	-	√
174	Road-U/C Edge	4064	24	2	0	3	----	0	-	√
175	Road-U/C Edge -- Clx	4065	24	2	0	2	----	0	-	√
176	Road-U/C Centreline -- Clx	4066	24	2	2	1	----	2	-	√
177	Road-U/C Centreline -- Sym	4067	24	2	0	5	----	0	-	√
178	Taxi/Bus Station	4068	20	3	1	0	4068	0	-	√
179	Track Edge	4069	25	4	0	3	----	0	-	√
180	Track Edge -- Clx	4070	25	4	0	2	----	0	-	√
181	Track Centreline -- Clx	4071	25	4	0	1	----	2	-	√
182	Track Centreline -- Sym	4072	25	4	0	5	----	0	-	√
183	Tunnel Portal	4073	19	3	2	0	----	0	-	√
184	Tunnel Edge	4074	19	5	0	3	----	0	-	√
185	Tunnel Edge -- Clx	4075	19	5	0	2	----	0	-	√
186	Tunnel Centreline -- Clx	4076	19	5	0	1	----	2	-	√
187	Tunnel Centreline -- Sym	4077	19	5	0	5	----	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
188	Driveway	4078	23	3	0	0	----	0	-	√
189	Electricity Line-High Voltage	5001	26	3	0	0	----	0	-	√
190	Electricity Line-Local	5002	26	9	0	0	----	2	-	√
191	Pipeline-Gas Above Ground	5003	27	4	1	0	----	0	-	√
192	Pipeline-Gas Below Ground	5004	27	10	1	4	----	0	-	√
193	Pipeline-Oil Above Ground	5005	28	6	1	0	----	0	-	√
194	Pipeline-Oil Below Ground	5006	28	12	1	4	----	0	-	√
195	Pipeline-Water Above Ground	5007	29	1	1	0	----	0	-	√
196	Pipeline-Water Below Ground	5008	29	7	1	4	----	0	-	√
197	Pole-Electric High Voltage	5009	26	3	1	0	5009	0	-	√
198	Pole-Electrical Local	5010	26	3	1	0	5010	0	-	√
199	Pole-Telephone	5011	30	0	1	0	5011	0	-	√
200	Pylon To Scale	5012	26	3	1	0	----	0	-	√
201	Pylon Symbol	5013	26	3	1	0	5013	0	-	√
202	Radio/Tv Antenna	5014	31	3	1	0	5014	0	-	√
203	Street Lamp-Multiple	5015	32	3	1	0	5015	0	-	√
204	Street Lamp-Left	5016	32	3	1	0	5016	0	-	√
205	Street Lamp-Right	5017	32	3	1	0	5017	0	-	√
206	Tank-Gas To Scale	5018	27	4	1	0	----	0	-	√
207	Tank-Gas Symbol	5019	27	4	1	0	5019	0	-	√
208	Tank-Oil To Scale	5020	28	6	1	0	----	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
209	Tank-Oil Symbol	5021	28	6	1	0	5021	0	-	√
210	Tank-Water To Scale	5022	29	5	1	0	----	0	-	√
211	Tank-Water Symbol	5023	29	1	1	0	5023	0	-	√
212	Telephone Box	5024	30	0	1	0	5024	0	-	√
213	Telephone Line	5025	30	0	0	0	----	2	-	√
214	Tower-Water To Scale	5026	29	7	1	0	----	0	-	√
215	Tower-Water Symbol	5027	29	1	1	0	5027	0	-	√
216	Traffic Light	5028	32	3	1	0	5028	0	-	√
217	Well-Oil	5029	28	6	1	0	5029	0	-	√
218	Well-Water	5030	29	1	1	0	5030	0	-	√
219	Manhole	5031	29	3	0	0	5034	0	-	√
220	Bench Mark-Ngn	6001	33	2	1	0	6001	0	-	√
221	Bench Mark	6002	34	2	1	0	6002	0	-	√
222	Control Point-Ngn	6003	33	3	1	0	6003	0	-	√
223	Control Point	6004	34	3	1	0	6004	0	-	√
224	Block Outline	6005	33	2	3	0	----	0	-	√
225	Parcel Outline	6006	34	0	1	0	----	0	-	√
226	Street Segment	6007	35	3	1	0	----	0	-	√
227	Boundary-Int'l Marked	6008	33	3	3	0	----	0	-	√
228	Boundary-Int'l Unmarked	6009	34	3	3	3	----	0	-	√
229	Beach-Shingle Pattern	7001	35	4	1	0	7001	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
230	Beach-Shingle Polygon	7002	35	10	0	0	----	2	-	√
231	Breakwater, Pier, Quay	7003	36	0	2	0	----	0	-	√
232	Canal-Edge	7004	37	1	1	0	----	0	-	√
233	Canal-Centreline	7005	37	7	1	7	----	0	-	√
234	Canal> 25m Centreline	7006	37	7	3	0	----	0	-	√
235	Canal 10-25m Centreline	7007	37	7	2	0	----	0	-	√
236	Canal5-10m Centreline	7008	37	7	1	0	----	0	-	√
237	Coastal Rock	7009	35	0	1	0	----	0	-	√
238	Coastline-Definite	7010	38	1	1	0	----	0	-	√
239	Coastline-Indefinite	7011	38	1	1	3	----	0	-	√
240	Dam	7012	36	3	2	0	----	0	-	√
241	Dam Interior Pattern	7013	36	9	0	0	----	0	-	√
242	Ditch	7014	37	1	1	5	----	0	-	√
243	Flow Arrow	7015	39	1	1	0	7015	0	-	√
244	Flow Arrowhead	7016	39	1	1	0	7016	0	-	√
245	Harbour Symbol	7017	36	1	1	0	7017	0	-	√
246	Lake Or Pond-Perennial	7018	38	7	1	0	----	0	-	√
247	Lake Or Pond-Dry	7019	38	7	1	2	----	0	-	√
248	Lake Or Pond Symbol	7020	38	7	1	0	7020	0	-	√
249	Levee	7021	36	0	1	6	----	0	-	√
250	Marsh Or Swamp Pattern	7022	40	7	1	0	7022	0	-	√

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		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
251	Marsh Or Swamp Polygon	7023	40	7	0	0	----	2	-	√
252	Mud Flat Pattern	7024	35	4	3	0	7024	0	-	√
253	Mud Flat Polygon	7025	35	10	0	0	----	2	-	√
254	Reservoir-Perennial	7026	38	3	1	0	----	0	-	√
255	Reservoir-Dry	7027	38	3	1	2	----	0	-	√
256	River-Perennial	7028	40	1	1	0	----	0	-	√
257	Stream-Perennial	7029	41	7	1	0	----	0	-	√
258	Wadi Large Outline	7030	41	1	1	3	----	0	-	√
259	Wadi Pattern	7031	41	7	1	0	7031	0	-	√
260	Wadi Small	7032	41	1	0	6	----	0	-	√
261	Water At Time of Air Photo	7033	38	7	0	0	----	0	-	√
262	Wadi Polygon	7034	41	1	0	0	----	2	-	√
263	Cliff	8001	42	0	1	0	----	0	-	√
264	Contour-Index	8002	43	6	2	0	----	0	-	√
265	Contour-Intermediate	8003	44	5	0	0	----	0	-	√
266	Contour-Index Depression	8004	43	6	2	3	----	0	-	√
267	Contour-Intermdt Depression	8005	44	5	0	3	----	0	-	√
268	Contour-Index Approximate	8006	45	6	2	4	----	0	-	√
269	Contour-Intermediate Approx	8007	46	5	0	4	---	0	-	√
270	Dem-Breakline-Round	8008	47	2	0	0	----	2	-	√
271	Dem-Breakline-Sharp	8009	47	3	0	0	----	2	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
272	Dem-Breakline-Structure	8010	47	4	0	0	----	2	-	√
273	Dem-Grid Point	8011	48	2	1	0	8011	2	-	√
274	Dem-Random Point	8012	48	3	1	0	8012	2	-	√
275	Spot Elevation	8013	48	6	1	0	8013	0	-	√
276	Spot Elevation Rooftop	8014	48	1	1	0	8014	0	-	√
277	Dem Exclusion Area	8015	47	1	0	0	----	2	-	√
278	Dem Approximate Area	8016	47	0	0	0	----	2	-	√
279	Dem Fault Line Top	8017	47	0	2	0	----	2	-	√
280	Dem Fault Line Bottom	8018	47	0	3	0	----	2	-	√
281	Cultivated Area Outline	9001	49	2	0	2	----	0	-	√
282	Cultivated Area Pattern	9002	49	8	1	0	9002	0	-	√
283	Cultivated Area Polygon	9003	49	8	0	0	----	2	-	√
284	Garden Outline	9004	50	2	0	2	----	0	-	√
285	Garden Pattern	9005	50	8	1	0	9005	0	-	√
286	Garden Polygon	9006	50	8	0	0	----	2	-	√
287	Gravel Area Pattern	9007	51	10	1	0	9007	0	-	√
288	Gravel Area Polygon	9008	51	10	0	0	----	2	-	√
289	Hedge	9009	15	2	1	7	----	0	-	√
290	Orchard Outline	9010	52	2	0	2	----	0	-	√
291	Orchard Pattern	9011	52	8	1	0	9011	0	-	√
292	Orchard Polygon	9012	52	8	0	0	----	2	-	√

S.#	FeatureNames	Feature Code/ Graphic Group/ Catro Line Style	Levels / Layers	Color	Weight	Line Style	Cell / Symbol / Block	Class / Construction	Font Tx / Size	Global
		ASC	LV	CO	WT	LC	CEL	CLA	FNT	SCO
293	Rock Outcrop	9013	51	6	0	2	----	0	-	√
294	Rock Outcrop Symbol	9014	51	12	1	0	9014	0	-	√
295	Rock Outcrop Polygon	9015	51	12	0	0	----	2	-	√
296	Palm Grove Outline	9016	52	6	0	2	----	0	-	√
297	Palm Grove Pattern	9017	52	12	1	0	9017	0	-	√
298	Palm Grove Polygon	9018	52	12	0	0	----	2	-	√
299	Sand Area Pattern	9019	51	3	1	0	9019	0	-	√
300	Sand Area Polygon	9020	51	3	0	0	----	2	-	√
301	Sand Dune Area Pattern	9021	51	6	1	0	9021	0	-	√
302	Sand Dune Area Polygon	9022	51	6	0	0	----	2	-	√
303	Scrub And Brush Pattern	9023	53	8	1	0	9023	0	-	√
304	Scrub And Brush Polygon	9024	53	8	0	0	----	2	-	√
305	Tree-Landmark Non Palm	9025	53	2	1	0	9025	0	-	√
306	Tree-Landmark Palm	9026	53	2	1	0	9026	0	-	√
307	Tree-Roadside/Ornamental	9027	23	2	1	0	9027	0	-	√
308	Trees-Scattered nonPalm symbl	9028	54	8	1	0	9028	0	-	√
309	Trees-Scattered Palm symbol	9029	54	8	1	0	9029	0	-	√
310	Trees-Scattered Polygon	9030	54	8	0	0	----	2	-	√

Appendix B - Glossary of Terms

Accuracy - The degree of agreement between a measurement, calculation, or specification and an accepted reference value or a standard.

Almanac - It is included in the navigational message transmitted by GNSS satellites, contains less accurate orbital information than ephemerides and is valid for a period of up to 90 days.

Antenna Phase Center - The apparent source of radiation on a GNSS receiver. The position of the antenna phase center is not necessarily the geometric center of the antenna.

Baseline - A GNSS baseline is established by two survey-quality GNSS receivers, with one at each end of the line to be measured, occupying and collecting GNSS data on two points during the same time - period.

Benchmark - A point marked with a survey mark, serving as elevation reference, a vertical control point.

Breaklines - Linear strings compiled along the breaks or changes of grade in the terrain, identifying terrain discontinuities such as ridges, toe of slopes, or any abrupt change in the existing surface.

Building Information Model - An Information model that represents structural elements using 3D digital technology while accommodating them with a data source that contains various related information about those elements.

Calibration - The process of changing the inputs or configuration of a device or system to improve the accuracy and precision of the estimate.

Confidence Level - Describes the level of statistical confidence in an estimated parameter or a test of significance. A 95 percent confidence level is common for most transportation operations.

Continuously Operating Reference Stations - A network of stations that provide global navigation satellite system data consisting of carrier phase and code range measurements in support of three-dimensional positioning.

Datum - A reference frame for precisely representing the positions and heights of locations on the surface of the Earth along with a set of reference points of known positions.

Digital Elevation Model - A digital model that describes elevations of the bare earth's surface, without including any natural or human-made feature that may lie on it.

Digital Surface Model - A digital model that describes elevations of the earth's surface, including its natural and human-made features.

Digital Terrain Model - A digital model that describes elevations of the bare earth's surface, without including any natural or human-made feature that may lie on it, augmented with breaklines.

Digital Twin - A digital representation of a planned or actual real-world physical infrastructure system that serves as the effectively identical digital counterpart of it for practical purposes, such as simulation, integration, testing, monitoring, and maintenance.

Ephemeris - It is included in the navigational message transmitted by GNSS satellites, is used for real-time satellite coordinate computation, and contains information on week number, satellite accuracy and health, age of data, satellite clock correction coefficients and orbital parameters.

Geodesy - A discipline that deals with measurement and representation of the Earth, including its gravity field in a three dimensional time varying space.

Geoid - The equipotential surface of the Earth's gravity field which best fits, in a least squaressense, global mean sea level at certain epoch.

Global Navigation Satellite System - Refers to several constellations of satellites broadcasting signals from space that transmit positioning and timing data to GNSS receivers which then use this data to determine their location.

Global Positioning System - Global Positioning System is a system of satellites, computers, and receivers that is able to determine the latitude and longitude of a receiver on earth by calculating the time difference for signals from different satellites to reach the receiver.

Integer Ambiguity - In GNSS surveys, is the unknown number of full wavelength cycles existing between a GNSS satellite transmitting a signal and the receiver that receives it.

International Federation of Surveyors - The premier international organization representing the interests of surveyors worldwide.

International Organization for Standardization (ISO) - An international nongovernmental organization of national standards bodies, headquartered in Geneva, Switzerland that develops and promotes worldwide technical, proprietary, industrial and commercial standards.

Least Squares Adjustment - A model for the solution of an overdetermined system of equations based on the principle of least squares of observation residuals. It is used extensively in the disciplines of surveying, geodesy, and photogrammetry.

Level - An optical instrument used to establish or verify points in the same horizontal plane in a process known as leveling, and is used in conjunction with a leveling staff to establish the relative height levels of objects or marks.

Level Staff (or Leveling Rod) - A graduated wooden or aluminium rod, used with a leveling instrument to determine the difference in height between points or heights of points above a vertical datum.

Light Detection and Ranging - A remote sensing method that uses light pulses from a laser to examine the surface of the earth and collect information about shape and surface characteristics. LiDAR can be used to make digital 3-D representation models of objects or areas on the earth.

Mobile Mapping Systems - Devices that collect geospatial data from a mobile platform, typically fitted with a range of cameras, LiDAR, Radar, echo-sounders, or any number of remote sensing systems, for land, air, or marine applications.

Photogrammetry - The science and technology of obtaining reliable information about physical objects and the environment through the process of recording, measuring, and interpreting photographic images and patterns of electromagnetic radiant imagery and other phenomena.

Precision - The degree of arrangement between measurements to each other.

Real-Time Kinematic - The application of surveying to correct common errors in current satellite navigation (GNSS) systems and relies on a reference station to provide real-time corrections, providing up to centimeter-level accuracy.

Right-of-Way (ROW) - The legal right, established by grant from a landowner or long usage (i.e., by prescription), to pass along a specific route through property belonging to another.

Side Slope - The slope from the edge of the roadway to the toe of a ditch, expressed as the ratio of horizontal distance to vertical distance.

Signal-to-Noise Ratio - A measure used in science and engineering that compares the level of the desired signal to the level of background noise. A ratio higher than 1:1 indicates more signal than noise.

Sound Navigation and Ranging - A technique that uses sound propagation (usually underwater, as in submarine navigation) to navigate, measure distances (ranging) and communicate with or detect objects on or under the surface of the water, such as other vessels.

Stereoplotting - The plotting of a map from aerial photographs using a stereoscopic device.

Survey Measurements - A process of measuring horizontal and vertical distances between objects, angles between lines, relative to either a horizontal or vertical plane, and slope distances.

Survey Work - The action of assessing and recording details about an area of land by determining the terrestrial or three-dimensional positions of points and the distances and angles between them. In order to provide assistance in planning construction projects.

Surveying - The science and art that aims at representing the Earth’s surface, including its horizontal and vertical features, on maps with appropriate drawing scales.

Surveyor - Responsible for providing information relevant to the shape and contour of the Earth's surface for engineering, mapmaking, and construction projects.

Total Point Station - An electronic/optical instrument used for surveying and building construction, that includes a theodolite to measure both vertical and horizontal angles, integrated with electronic distance measurement to measure the slope distance from the instrument to a particular point, and an on-board computer to collect data and perform calculations.

Traverse - A method in the field of surveying to establish control networks by placing survey stations along a line or path of travel, and then using the previously surveyed points as a base for observing the next point.

Triangulation - The process of determining the location of a point by forming triangles to the point from known points involving only angle measurements at known points.

Trilateration - The process of determining the location of a point by forming triangles to the point from known points involving distance measurements at known points.

Appendix C - Abbreviations, Acronyms

3D	Three Dimensional
AAA	Ain Al Abd Datum
AI	Artificial Intelligence
AMG	Automatic Machine Guidance
AOI	Area of Interest
APC	Antenna Phase Center
ARAMCO	Saudi Arabian Oil Company
ARP	Antenna Reference Point
ASCII	American Standard Code for Information Interchange
ASPRS	American Society of Photogrammetry and Remote Sensing
AUV	Autonomous Underwater Vehicle
BA	Bundle Adjustment
BIM	Building Information Model
BOSSI	Board of Surveying and Spatial Information
BSQ	Band Sequential
BVLOS	Beyond Visual Line-of-Sight
CADD	Computer-Aided Design and Drafting
CALTRANS	California Department of Transportation (USA)
CL	Confidence Level
CMOS	Complementary Metal-Oxide Semiconductor
CORS	Continuously Operating Reference Stations
DEM	Digital Elevation Model
DGNSS	Differential GNSS
DMAT	Department of Municipal Affairs and Transport

DMI	Distance Measurement Instrument
DSM	Digital Surface Model
DTM	Digital Terrain Model
DXF	Drawing Exchange Format
EDM	Electronic Distance Measurement
EPSG	European Petroleum Survey Group
FGCC	Federal Geodetic Control Commitee
FIG	International Federation of Surveyors
FoV	Field of View
GASGI	General Authority for Survey and Geospatial Information, KSA (former GCS)
GCP	Ground Control Point
GCS	General Commission for Survey, KSA (name of GASGI before September 2020)
GDMS	Ministry of Defense - General Directorate of Military Survey
GDOP	Geometric Dilution of Precision
GIS	Geographic Information System
GLN	Geodetic Leveling Network
GLONASS	Global'naya Navigatsionnaya Sputnikovaya Sistema
GNSS	Global Navigation Satellite System
GOCE	Gravity Field and Steady-State Ocean Circulation Explorer
GPS	Global Positioning System
GRF	Geodetic Reference Frame
GRS	Geodetic Reference System
GSD	Ground Sampling Distance
GSM	Global System for Mobile Communication

ICP	Iterative Closest Point
IGS	International GNSS Service
IMU	Inertial Measurement Units
INS	Inertial Navigation System
ISO	International Organization for Standardization
ITRF	International Terrestrial Reference Frame
ITRS	International Terrestrial Reference System
JPEG	Joint Photographic Experts Group
KACST	King Abdulaziz City for Science and Technology
KSA	Kingdom of Saudi Arabia
KSA-CORS	Kingdom of Saudi Arabia Continuously Operating Reference Stations
KSA-GEOD	KSA Geoid Model
KSA-GRF	Kingdom of Saudi Arabia Geodetic Reference Frame
KSA-VRF	Kingdom of Saudi Arabia Vertical Reference Frame
LiDAR	Light Detection and Ranging
LOD	Level of Development
MBES	Multi-Beam Echo-Sounders
MEMS	Micro-Electro-Mechanical Systems
MMS	Mobile Mapping System
MoMRAH	Ministry of Municipal, Rural Affairs and Housing
MoTLS	Ministry of Transport and Logistic Services
MTRF	MOMRA Terrestrial Reference Frame
MVS	Multi View Stereo
MXD	Map Exchange Document
NGN	National Geodetic Network

NGS	NOAA’s National Geodetic Survey
NOAA	National Oceanic and Atmospheric Administration
NSW	New South Wales (Australia)
NVRS	National Vertical Reference System
PC	Point of Curvature
PDF	Portable Document Format
PDOP	Position Dilution of Precision
PI	Point of Intersection
PPK	Post Processing Kinematic
PPM	Parts Per Million
PPP	Precise Point Positioning
PT	Point of Tangency
QA	Quality Assurance
QC	Quality Control
RINEX	Receiver Independent Exchange Format
RMSE	Root Mean Square Error
ROV	Remotely Operated Vehicles
ROW	Right-of-Way
RTK	Real Time Kinematic
SANSRS	Saudi Arabia National Spatial Reference System
SBES	Single-Beam Echo-Sounders
SfM	Structure from Motion
SGN	Saudi Geodetic Network
SHC	Saudi Highway Code
SLAM	Simultaneous Localization and Mapping

SNR	Signal-to-Noise Ratio
SONAR	Sound Navigation and Ranging
TBM	Tunnel Boring Machine
TGBM	Tide Gauge Benchmark
THU	Total Horizontal Uncertainty
TIFF	Tag Image File Format
TIN	Triangulated Irregular Network
TLS	Terrestrial Laser Scanning
ToF	Time of Flight
TPS	Total Point Station
TPU	Total Propagated Uncertainty
TVU	Total Vertical Uncertainty
UAV	Unmanned Aerial Vehicles
UHF	Ultra-High Frequency
USV	Unmanned Surface Vehicle
UTM	Universal Transverse Mercator
vSLAM	Visual Simultaneous Localization and Mapping
WGS	World Geodetic System

Appendix D - Units

SI Units		Imperial Units	
Length			
nm	Nanometer		
mm	Millimeter		
cm	Centimeter		
m	Meter		
km	Kilometer		
Area			
mm^2	Square millimeter		
m^2	Square meter		
km^2	Square kilometer		
Weight			
kg	Kilogram		
Time			
s	Second		
min	Minute		
h	Hour		
week	Week		
month	Month		
Angular			
°	Degree		
Other			
km/h	Kilometers per hour		
°/h	Degrees per hour		
MHz	Megahertz		
kHz	Kilohertz		
m/s	Meters per second		

SI Units		Imperial Units	
mm/m	Millimeters per kilometer		
mm/km	Millimeters per kilometer		
Other			
ppm		Parts per million	
dpi		Dots per inch	

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