MINISTRY OF SCIENCE AND HIGHER EDUCATION OF THE RUSSIAN FEDERATION

Federal State Autonomous Educational Institution of Higher Education "National Research Technological University MISiS" branch in Almalyk

TRAINING AND METODOLOGY COMPLEX
in the discipline "Geodesy"

Direction of preparation: **05/21/04 - "Mining"**

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The educational and methodological complex for the discipline "Geodesy" was compiled in accordance with the requirements of the educational standards of NUST "MISiS" on the basis of curricula in the relevant areas of training and is a presentation of the questions "The subject and tasks of geodesy, the concept of the figure of the Earth, the image of the surface of an ellipsoid on a plane, topographic plans and maps, cartographic projections, tasks solved on a topographic map, orientation of lines, geodetic measurements, topographic surveys, information from the theory of measurement errors and modern methods of engineering and geodetic work"

For students of universities of mining and metallurgical, engineering and construction specialties and specialties of the land cadastre.

Most of the topics presented in the academic work are basic in the study of all geodetic courses ("Fundamentals of Topography", "Engineering Geodesy"). Therefore, the teaching materials can also be useful for students of other specialties.

1 PLANNED LEARNING OUTCOMES IN THE DISCIPLINE

1.1. The purpose and objectives of the discipline

The discipline "Geodesy" refers to the compulsory discipline of the basic part of the professional cycle of disciplines and was developed in accordance with the OS VO NUST "MISiS": Educational standard of higher education in the field of study **21.05.04.**

The purpose of the discipline : Obtaining theoretical knowledge by students in the field of mapping the earth's surface and acquiring practical skills in the production of angular and linear measurements on the ground in order to depict them on plans, maps and profiles.

Discipline tasks:

Objectives of the discipline: - to master the theoretical foundations of the methods and techniques of measurement on the earth's surface, the study of the shapes and sizes of the Earth. Mastering the methods and methods of measurements performed by geodetic instruments and devices, determining the position of individual points on the earth's surface in the selected coordinate system, compiling maps and plans of the area for various purposes, performing measurements on the earth's surface necessary for the design, construction and operation of engineering structures, the operation of natural wealth of the Earth's surface and its interior.

Master the methods of creating maps, phototopography (photogrammetry), maps and plans from aerospace images. To master the methods of geodetic work in surveys in the construction and operation of engineering structures during the installation of complex equipment, in the exploitation of natural resources.

1.2. Planned learning outcomes

The discipline "Geodesy" refers to the basic part of the professional cycle of disciplines and is mandatory for the development of the PEP in the specialty "Mining".

The ability to select and (or) develop the provision of integrated technological systems for operational exploration, production and processing of solid minerals, as well as enterprises for the construction and operation of underground facilities with technical means with a high level of automation of control

As a result of mastering the discipline, graduates will:

Know:

-theoretical foundations of methods and techniques for mapping the earth's surface. The principles of the theodolite device, measuring angles; leveling devices, the basics of leveling the surface. Know the existing methods for performing basic geodetic work.

-existing methods for performing basic geodetic works

-possession of skills in analyzing mining and geological conditions during operational exploration and production of solid minerals, as well as during the construction and operation of underground facilities

- coordinate systems used in Geodesy.
- basic methods for drawing up geodetic projects for solving applied problems of geodesy in mining, construction and operation of various engineering structures.

Be able to:

- determine the spatial and geometric position of objects, carry out the necessary geodetic and mine surveying measurements, process and interpret their results

- demonstrate a deep knowledge and understanding of the fundamental sciences, as well as knowledge in interdisciplinary areas of professional activity
- skills in analyzing mining and geological conditions during operational exploration and production of solid minerals, as well as during the construction and operation of underground facilities
- determine the coordinates of points according to plans and maps.
- choose methods and tools for performing geodetic measurements and programs for processing measurement results.
- determine the spatial and geometric position of objects, carry out the necessary geodetic and mine surveying measurements, process and interpret their results
- choose methods and tools for performing geodetic measurements and programs for processing measurement results.
- demonstrate a deep knowledge and understanding of the fundamental sciences, as well as knowledge in interdisciplinary areas of professional activity
- apply knowledge in mathematics to solve geodesic problems.
- determine the spatial and geometric position of objects, carry out the necessary geodetic and mine surveying measurements, process and interpret their results.

Own:

- methods of using graphic documentation for solving engineering problems.
- demonstrate a deep knowledge and understanding of the fundamental sciences, as well as knowledge in interdisciplinary areas of professional activity
- basic knowledge in the field of mathematics and physics, and find the relationship with geodesy.
- methods to select and develop in order to provide integrated technological systems for operational exploration, production and processing of solid minerals, as well as enterprises for the construction and operation of underground facilities with technical means with a high level of control automation
- methods of using graphic documentation for solving engineering problems.
- skills in analyzing mining and geological conditions during operational exploration and production of solid minerals, as well as during the construction and operation of underground facilities
- methods of selection of equipment, tools and programs for processing measurement results to perform geodetic surveys.

1.3 Competences formed by the discipline

The study of the discipline "Geodesy" is aimed at the formation of professional competencies among students, the possession of which can be identified on the basis of students' manifestation of abilities:

- the ability to select and (or) develop the provision of integrated technological systems for operational exploration, production and processing of solid minerals, as well as enterprises for the construction and operation of underground facilities with technical means with a high level of automation of control
- the ability to determine the spatial and geometric position of objects, carry out the necessary geodetic and mine surveying measurements, process and interpret their results

-possession of skills in analyzing mining and geological conditions during operational exploration and production of solid minerals, as well as during the construction and operation of underground facilities

- the ability to determine the spatial and geometric position of objects, to carry out the necessary geodetic and mine surveying measurements, to process and interpret their results
- the ability to demonstrate a deep knowledge and understanding of the fundamental sciences, as well as knowledge in interdisciplinary areas of professional activity

2. THE PLACE OF DISCIPLINE IN THE STRUCTURE OF THE EDUCATIONAL PROGRAM

Requirements for the preliminary preparation of the student:

Geology
Maths
Physics

Disciplines (modules) and practices for which the development of this discipline (module) is necessary as a previous one:

Defense of the final qualifying work
Geology
Mining Fundamentals
Open Geotechnology
underground geotechnology
Practice for obtaining primary professional skills and abilities
Construction Geotechnology
Educational practice (geological)
mine surveying
Metrology and standardization
Subsoil geometry
mining ecology
Mine Surveying and Geodetic Instruments
Mathematical processing of measurement results
Industrial practice (technological practice, practice for obtaining primary professional skills and abilities)
Geomechanical support of mining operations
Geomechanical support of open pit mining
Geomechanical support of underground mining
Geophysical methods for studying deposits
Mineral enrichment
Industrial sanitation and occupational health
Rational use and protection of natural resources
Information modeling technologies in construction
Subsoil modeling
Reclamation of disturbed lands
Analysis of the accuracy of surveying work
Mining logistics
Modeling and calculation of underground structures
Protection and rational use of subsoil
Reconstruction of mining enterprises
Movement and deformation of rock masses and the earth's surface
Technology and safety of blasting
Mining safety assurance of mining operations
Open pit mining during construction
Managing the state of the rock mass
Stability management of sloping structures
Audit and expertise of industrial safety
Unmanned technologies in mine surveying
Subsoil geodynamics

3.VOLUME OF DISCIPLINE AND TYPES OF EDUCATIONAL WORK

The total labor intensity of mastering the discipline "Geodesy" is 92.3 hours.

72 hours are allocated for contact work (classroom studies) of students with a teacher, including 36 hours for lectures, 18 hours for practical classes and 18 hours for laboratory classes. On control 20.3 hours.

The program provides for three independent work, 2 tests (intermediate control) and a final test with an assessment.

4. CONTENT OF THE DISCIPLINE BY TYPE OF ACTIVITIES

Chapter 1
Determining the position of points on the earth's surface

§ 1 . _ A Brief Historical Outline of the Development of Geodesy .
§ 2. The concept of the shape and size of the Earth.
§ 3. Ellipsoid of FN Krasovsky.
§ 4. The principle of depicting the earth's surface on a plane.
§ 5. Heights of points on the earth's surface.

Chapter 2
The concept of the coordinate system used in geodesy

§ 6. Geographic coordinate system. Meridians and parallels. Geographic latitude and longitude.
§ 7. System of plane rectangular coordinates. Gauss-Kruger projection. Distribution of six-degree zones. Zonal system of rectangular coordinates. Reshaped ordinates. coordinate grid.
§ 8. Influence of curvature of the earth on the measured distances and heights of points.

Chapter 3
Orienting lines and orienting angles

§ 9. Orientation of lines. True (geographical) meridian. magnetic meridian. Axial meridian of the zone
§ 10. Orienting angles. Geographic (true) azimuth. Magnetic azimuth. Directional angle. Rumb.
§ 11. Connection between orienting angles.

Chapter 4
Basic survey drawings

§ 12. The concept of a plan and a map.
§ 13. Scales.
§ 14. Conventional signs and their classification.
§ 15. Representation of relief on plans and maps.
§ 16. Nomenclature of maps and plans.

Chapter 5
Practical use of the map (plan)

§ 17. Determining the coordinates of points on the map. Determination of geographical coordinates. Definition of rectangular coordinates.
§ 18. Orientation of lines. Determination of the directional angle of the line, geographic and magnetic azimuths.
§ 19. Determination of the altitude position of points. Interpolation.
§ 20. The steepness of the slope.
§ 21. Laying schedule.
§ 22. Deviations.
§ 23. Building a terrain profile.
§ 24. Determining areas according to the plan.

Chapter 6
Angle measurement

§ 25. The principle of measuring angles on the ground.

Chapter 12

§ 54. Ways of leveling.
§ 55. The principle and methods of geometric leveling.
§ 56. Complex leveling.
§ 57. The device of the level.
§ 58. Checks and adjustments of the level.
§ 59. Trigonometric leveling

Chapter 13. Use of modern geodetic instruments in solving geodetic problems.

§ 60. Automatic theodolites and levels.
§ 61. Principles of operation of automatic geodetic instruments.
§ 62. Automatic system of theodolite survey.
§ 63. Automatic leveling system.

4.2 Practical exercises

No.	Topic of the lesson	Hours allotted
1	Working with a topographic map. Determination of areas on the map	2
2	Working with a topographic map. Relief drawing with contour lines based on numerical marks of points	2
3	Working with theodolite. Installation. Checks and adjustments.	2
4	Working with theodolite. Removal of reference on a horizontal circle. Measurement of horizontal angles.	2
5	Working with theodolite. Removal of the reference in a vertical circle. Measurement of vertical angles. Determining the place of zero MO.	2
6	Working with theodolite. Distance measurement	2
7	Level work. Installation. Checks and adjustments. Removing the reading on the rail.	2
8	Level work. Measurement of excesses. Double leveling method.	2
9	Level work. Method of leveling from the middle.	2
	Total:	**18**

4.3 Laboratory work

No.	Topic of the lesson	Hours allotted
one	Working with a topographic map	2
2	Working with a topographic map	2
3	Working with a topographic map	2
4	Working with theodolite. Installation. Checks and adjustments. Removal of reference on a horizontal circle. Measurement of horizontal angles.	2
5	Working with theodolite. Removal of the reference in a vertical circle. Measurement of vertical angles. Determining the place of zero MO.	2
6	Working with theodolite. Processing of theodolite survey materials and drawing up a site plan	2
7	Level work. Log Processing Track Leveling and Profiling	2
8	Level work. Log Processing Track Leveling and Profiling	2
9	Level work. Log Processing Track Leveling and Profiling	2
	Total:	**18**

5 EDUCATIONAL AND INFORMATION SUPPORT OF THE DISCIPLINE (MODULE)

5.1 Main literature:

1. Popov V.N., Bukrinsky V.A., Bruevich P.N., Geodesy and mine surveying: a textbook for high schools in - M: Mining book, 2010.
2. Popov V.N., Chekalin S.I., Geodesy: a textbook for students. universities according to special "Mine Surveying" eg. "Mining " - M .: Mining book, 2012
3. Kuleshov D.A., Strelnikov G.E., Ryazantsev G.E. Engineering geodesy - M. Kartgeocenter - Geodesizdat, 1996
4. Dementiev V.E. Modern geodetic equipment and its application .- Tver, LLC IPP "ALEN", 2006.
5. FlexLine_plus_User Manual. 66178-3.0.0en.Translated from the original text (766166-3.0.0en) Printed in Switzerland. © 2011 Leica Geosystems AG, Heerbrugg, Switzerland
6. Leica NA320/24/32_User Manual. Translation of the original text (8378 61-1.0.0en.) Printed in Switzerland. © 2015 Leica Geosystems AG, Heerbrugg, Switzerland

5.2. Additional literature:

7. Paramonov A.G., Chernoglazov N.V., Somov V.D. Fundamentals of topography and aerial photography. - M., Nedra, 1991
8. Glinsky S.P., Grechaninova G.I., Danilevich V.M. etc. Geodesy. - M.: Geodesizdat, 1995
9. Conventional signs for topographic plans at a scale of 1:5000,

1: 2000, 1:1000, 1:500.-M.; Nedra, 2000

10. Fedotov G.A. Engineering geodesy - M. "Higher School", 2004.

5.3. Information and telecommunication resources

one.	Determining the position of points on the earth's surface. The concept of coordinate systems.	https://youtu.be/IOoncSAY8WU https://youtu.be/3JAxwyeIMpU
2.	Orientation of lines on the ground	https://youtu.be/Y_I4EQmrWnY
3.	Basic geodetic drawings. The concept of the plan, map, profile and section. Practical use of the plan (map) for solving engineering and technical problems.	https://youtu.be/3TE8yIilgdk
four.	Angle measurement.	https://youtu.be/InZ5TpTPHVA https://youtu.be/eU8wQvbzw-4
5.	Leveling	https://youtu.be/P0wiHl0HXUk https://youtu.be/p5xEu_JkfqM

5.3 Logistics of discipline

The following are used in the educational process: - classrooms for conducting lecture and seminar-type classes, group and individual consultations, current control and intermediate certification, a set of multimedia equipment, including a multimedia projector screen, a computer for demonstrating presentations; educational visual aids providing thematic illustrations and classroom furniture (tables, chairs, classroom board); - the library for independent work is equipped with computer tables, chairs, classroom board, computer equipment with the ability to connect to the Internet and provide access to the university's electronic information and educational environment.

6. FUND OF EVALUATION FOR INTERIM CERTIFICATION

The system for assessing students' knowledge pursues the following tasks:
- organization of systematic and timely assimilation of the subject;
- regular assessment of students' progress;
- objective and accurate assessment of students' knowledge;
- regular notification of students and analysis of assessment results.

6.1. Types and forms of the rating system

Assessment of student progress in the subject is carried out regularly and is achieved by the following types of control:

- current control (TC);
- intermediate control (PC);
- final control.

Current control provides for the assessment of knowledge and practical skills on each topic of the discipline and is carried out in practical and laboratory classes.

Intermediate control assesses the theoretical knowledge of students after classes in 8 (PC-1) and 17 (PC-2) weeks and determines the student's ability to answer questions on the topics covered by the discipline. PC form - written test.

The final control is carried out throughout the course of the subject at the end of the semester.

The evaluation fund is formed on a 100-point scale. 100 points are distributed as follows:

- current control 50 points
- intermediate control (PC-1 + PC-2) 10 points
- final control 40 points

The following typical criteria for assessing students' knowledge are taken into account:

score	Grade	Student knowledge level
86-100	Excellent (5)	- decision making and conclusions; - the ability to think creatively; - independent thinking - the ability to apply the acquired knowledge in practice; - the concept of the essence of the issue, knowledge and ability to express; - have an idea about the subject being studied.
71-85	Good (4)	- independent thinking - the ability to apply the acquired knowledge in practice; - the concept of the essence of the issue; - the ability to express the acquired knowledge.
55-70	Satisfactory (3)	- the concept of the essence of the issue; - the ability to express the acquired knowledge; - to have an idea about the subject being studied.
0-54	Unsatisfactory	- not having an accurate representation; - lack of discipline

6.2 The procedure for assessing students' knowledge:

• delivery of works in practical and laboratory classes (description of work and its defense) - the maximum is estimated up to 50 points for the performance and defense of laboratory and practical works: work No. 1 - 20 points; work No. 2 -15 points, work No. 3 -15 points;

• **intermediate control work** (written) - each PC is estimated at a maximum of 5 points. During the semester, two intermediate tests are held (PC-1 and PC-2) at 8 and 17 weeks - the maximum you can score is 10 points;

• **final control work** - written work is estimated at a maximum of 40 points.

7. METHODOLOGICAL INSTRUCTIONS FOR STUDENTS ON MASTERING THE DISCIPLINE

The organization of classes in the discipline "Life Safety" is possible both according to the usual technology by type of work (lectures, practical classes, laboratory workshop, current control) according to the schedule, and via distance learning through the NUST MISIS AF platform.

When presenting theoretical material, multimedia illustrative materials are used.

Laboratory classes are conducted using laboratory equipment.

For independent work, educational and methodological materials prepared by teachers are used.

Training is organized in accordance with this program .. The protection of practical and laboratory classes by students is carried out during these works or out of class time. Three examinations to be carried out in writing ..

For independent work, students are provided with computer classes of the AF NUST MISiS. In the process of independent work, students use the computer classes of the AF NUST MISiS.

8. INFORMATION TECHNOLOGIES USED IN THE IMPLEMENTATION OF EDUCATIONAL ACTIVITIES IN THE DISCIPLINE

Lectures and practical classes are conducted using multimedia tools. Laboratory classes are held in laboratory classes in compliance with safety requirements. The current certification involves the delivery of topics of practical and laboratory classes.

9. LOGISTICS OF THE DISCIPLINE

9.1. Specialized laboratories and classes, main installations and stands
9.2. Means of ensuring the development of discipline (module)

10 LECTURES

Chapter 1

Determining the position of points on the earth's surface

§ 1. Brief historical outline of the development of geodesy .

Geodesy is a science that studies the figure and gravitational field of the Earth, the planets of the solar system, methods and methods for determining the position of points in the accepted coordinate system and is engaged in accurate measurements on the ground necessary to create maps and plans of the earth's surface, solve various production and technical problems of the national economy and defense of the country.

Scientific and practical problems of geodesy are solved on the basis of geodetic measurements using geodetic instruments.

In geodesy, the following sections are distinguished:

- higher geodesy;
- topography;
- the engineering geodesy.

Higher geodesy is a science that studies the shape of the Earth and its external gravitational field, determines the coordinates of individual points on the earth's surface in a single system, and also studies the horizontal and vertical movements of the earth's crust. This area of geodesy also deals with the study of the figures of the planets of the solar system and their gravitational fields.

In higher geodesy, to study the figure of the earth and its external gravitational field, a theory and methods are being developed:

- high-precision measurements on the earth's surface of distances and heights, horizontal and vertical angles between directions on the earth's surface;
- gravity measurements;
- observation of artificial satellites of the Earth in order to determine their position in near-Earth space;
- determination of geographical latitudes, longitudes and directions of meridians as a result of observation of satellites and astronomical bodies;
- obtaining quantitative characteristics of vertical and horizontal tectonic movements of the earth's crust.

Points on the surface of the Earth, the coordinates of which are determined by the methods of higher geodesy, form the state geodetic network. The state geodetic network is the basis for the survey network created during topographic survey.

Topography (from the Greek . place and write) - a scientific discipline that studies the earth's surface, i.e. elements of the physical surface of the land and the objects of human activity located on it in geometric terms, as well as ways to represent it.

The tasks of topography include:

- organization of measurements on the ground, processing of their results in order to create topographic maps (scales 1:100,000 and larger) and geographical study of the surveyed area;
- creation of a survey network on the ground, consisting of points, the position of which is determined in a single coordinate system;
- organization and execution of filming works by ground methods;
- organization and development of methods for performing survey work using materials from ground and aerial stereo photography.

The main type of survey for compiling topographic maps at present is aerial topographic survey - photographing the terrain from the air with subsequent processing of photographic images.

Engineering geodesy studies the methods of geodetic work performed during surveys, design, construction and operation of various engineering structures; in the exploration, development and exploitation of the natural resources of the country and its subsoil.

In engineering geodesy, methods of higher geodesy, topography, and photogrammetry are used.

Topography and aerial topography are engaged in the development of methods for creating plans and maps from photographs and aerial photographs of the area.

The field of science, technology and production that covers the study, creation and use of maps is called *cartography.*

The tasks of engineering geodesy and topography are solved with the help of special measurements. Measurements should be performed with the necessary, reasonable accuracy. Measurements made with overestimated accuracy lead to unnecessary expenditure of effort, money, time, and with insufficient accuracy - to errors and defects in work. Therefore, before measurements, an engineering calculation is carried out in order to select a method for obtaining results with a given accuracy.

Geodesy arose in ancient times, when for the development of human society it became necessary to measure the areas of cultivated fields, to study the earth's surface for economic purposes. In ancient Egypt in the 18th century. BC. there was a manual for solving arithmetic and geometric problems that arise when measuring and determining the areas of land. There is evidence that in China around the 10th c. BC. there was an institution for topographic surveys of the country. In Babylon and Assyria (7th century BC), general geographical and special maps were compiled on clay tablets. The construction of canals and irrigation systems could not be carried out without geodetic measurements and surveys of the terrain, performed with a sufficiently high accuracy.

After Pythagoras (580-500 BC) and Aristotle (384-322 BC) suggested that the Earth was spherical, Eratosthenes (276-195 BC)) were made determinations of the radius of the Earth. These definitions were based on a geometric method called degree measurements. In the 2nd century BC. astronomers and mathematicians established the concepts of geographic latitude and longitude of a place, developed the first cartographic projections with a grid of meridians and parallels on maps, and gave methods for determining the position of points on the earth's surface from astronomical observations.

The first information about the performance of geodetic measurements in Russia dates back to 1068, when, on the orders of Prince Gleb, between the cities of Kerch and Taman, the width of the Kerch Strait was measured on ice. The collection of laws of Ancient Russia " Russian Truth", referring to the 11-12 centuries, contains decrees on land boundaries, which were established by measurements on the ground. During the reign of Ivan the Terrible (1530-1584), "service people" were required to take pictures and make a description of the places where they were going. Based on these materials, a map of the Moscow state was drawn up on a scale of 1: 1,800,000, known as the Big Drawing.

The development of geodesy and geodetic works in Russia intensified under Peter I. In 1701, he founded in Moscow one of the first astronomical observatories in Russia and the School of Mathematical and Navigational Sciences, which trained astronomers, geodesists, geographers, hydrographers and navigators. In 1701 S.W. Remizov and his sons compiled the Drawing Book of Siberia, which was the first Russian geographical atlas (23 maps). In 1745, the Russian Atlas was published at the Academy of Sciences. In 1797, a map depot was organized at the General Staff of the Army. In 1822, the Corps of military topographers was created. All major astronomical-geodesic and topographic works on the territory of Russia in the 19th and early 20th centuries. carried out by the specialists of this institution. As a result of the work of the Corps of Military Topographers, topographic maps of the border regions of Russia were created on a scale of 1 and 2 versts per inch, a 3-verst map of Western Siberia, a 10-verst map of the European part of Russia and Western Russia.

In the Soviet period, surveyors carried out a huge amount of work that met the needs of socialist construction. Soviet scientist M.S. Molodensky developed a new theory for studying the figure of the Earth and its external gravitational field, which put Soviet geodesy in first place in the world in terms of solving its main problem. Soviet surveyors under the leadership of F.N. Krasovsky, new parameters of the figure of the Earth were obtained. In its development, geodetic sciences rely on the achievements of other scientific disciplines, primarily physics, higher mathematics, and electronics. A new era in the development of geodesy opened with the launch in the USSR of the world's first artificial Earth satellite. Satellite systems for determining the location of various objects, surveying the Earth's surface from space, the widespread use of radio-electronic, laser and optical electronic systems and devices make it possible to perform geodetic measurements, topographic and special surveys in a new way quickly and with high accuracy.

§ 2. The concept of the shape and size of the Earth.

Geodesy and topography play a special role in solving various problems in the national economy, for example, in the exploration, design and construction of large hydraulic structures, industrial complexes, railways and roads, airfields, cities and towns, ground and underground communications, pipelines, in the development deposits of various natural resources.

Topographic and geodetic work is an integral part of the search, exploration and development of oil and gas fields. Recently, the volume of geological and geodetic research on the development of offshore oil and gas fields has been increasing. The planning and implementation of these works is impossible without the use of geodetic methods and modern automated navigation and geodetic systems, which make it possible to quickly determine the position of research ships and aircraft located at a great distance from the coast with the necessary accuracy.

During the operation of oil and gas fields, deformations of the earth's surface occur on the territory of these fields. Data allowing to observe the dynamics of this process are obtained from repeated geodetic observations of a network of special points.

Topographic maps - detailed, unified in content, design and mathematical basis; geographic maps displaying the main natural and socio-economic objects (relief, vegetation, settlements, roads, economic objects, etc.) are created on a single geodetic basis. The value of topographic maps can hardly be overestimated - they make it possible to study the area without direct observation of it and are the basis on which the results of research in geology, geophysics, geomorphology and other sciences that study the Earth are displayed. Based on topographic maps, geographic, geological and other special maps are compiled. Topographic maps are used in state planning, for the design of engineering structures, in the exploration of natural resources, and in the organization of the exploitation of oil, gas and mineral deposits.

Topographic maps of different years, supplemented by materials from remote surveys in different zones of the spectrum of electromagnetic oscillations, are used in the study of processes occurring in nature under the influence of human economic activity, to solve environmental issues.

When searching for and exploring oil and gas fields, topographic maps are used for preliminary study of the territory in order to plan geological and geophysical studies. In the course of prospecting and exploration geological and geophysical works, topographic maps are necessary for orientation on the ground and for tying points of geological research. According to the topographic map, geophysical routes (profiles) are designed, and then, using geodetic methods, the position of these routes is indicated on the ground.

During the development of an oil or gas field, geodetic methods determine the position of all wells, surface and underground structures, pipelines, power lines and communications. Then these objects are put on a map or plan of the field.

In the process of aerial survey, aerial photographs of the earth's surface are obtained with high image quality. These images can be used not only for the preparation of topographic maps, but also for the geological study of the territory.

The characteristic of aerial photography as a method of studying natural resources was given by Academician A.E. Fersman in 1928. Aerial photography provides an accurate and objective photographic image of the territory, it allows you to repeat the shooting in different periods and establish the changes that nature and human economic activity make over a certain period of time.

Space images provide a large amount of information on the structural structure of territories. They can be used to identify folded structures and faults of various orders: from global and regional, not traced by aerial photographs, to local, with dimensions of several kilometers.

Prominent Soviet geologist Academician I.M. Gubkin, noting the importance of topographic work, wrote that topographic maps and plans serve as a necessary condition for the subsequent successful implementation of the work of a geologist, prospector, hydrologist, hydraulic engineer, geographer, soil scientist, forestry specialist, design engineer, builder, etc.

The first in the history of science to determine the size of the Earth, as a ball, was carried out in ancient Egypt by Eratosthenes. In the second half of the 17th century. centrifugal force was discovered and the dependence of the period of oscillation of a physical pendulum on its length and acceleration of gravity was discovered. The facts of the change in the length of the second pendulum with the change in the latitude of the place were established. The generalization of this information and the discovery of the law of universal gravitation led to the assumption that the Earth is oblate in the direction of the poles.

To test this assumption, the Paris Academy of Sciences organized expeditions to Peru and Lapland (1735-1742) to perform degree measurements. The results of these measurements confirmed the oblateness of the Earth in the direction of the poles and gave one more proof of the validity of the law of universal gravitation.

By the middle of the 18th century. the French mathematician Clairaut derived a second-order differential equation relating the density and compression of the inner spheroidal layers of the Earth. Clairaut's differential equation, subsequently properly refined, is now used to determine the compression of the Earth based on the assumption of its internal structure. Thus, the law of the distribution of gravity on the surface of the earth's ellipsoid was discovered and a connection was established between the compression of the earth's ellipsoid and the distribution of gravity on its surface.

In 1785 in France A.M. Legendre introduced the concept of a potential function, which marked the beginning of the development of potential theory and is of great importance for geodesy in studying the figure of the Earth.

In 1792-1799, P. Mechain and J. Delambre (France) measured the arc of the meridian from Dunkirk to Barcelona to establish the length of a meter as 1/10,000,000 of a quarter of the earth's meridian. Based on the results of these measurements, for the first time, the dimensions of the earth's ellipsoid were determined with sufficient accuracy.

In 1816, the degree measurement began and ended in 1855 in Russia. The work was headed by the famous astronomer V.Ya. Struve. The length of the measured arc of the meridian from the mouths of the Danube to the shores of the Arctic Ocean was 25° in latitude.

In addition to the Struve arc in Russia in 1848-1858. degree measurements were carried out along the 48 ° parallel from Chisinau to Astrakhan with a length of about 20 °, and in 1861-1870. along the 52° parallel from the western borders to Orsk, with a length of about 39° in longitude. According to these degree measurements in 1893, A.M. Zhdanov produced one of the well-known measurements of the dimensions of the earth's ellipsoid. Previously, scientists from different countries, such as Bessel, Clark, Hayford, determined the parameters of the earth's ellipsoid.

In 1828, Gauss proposed to take for the mathematical surface of the Earth the level surface of the potential of gravity, coinciding with the mean sea level. The discrepancies in the results obtained by different authors, discovered in the course of determining the parameters of the earth's ellipsoid, showed that the figure of the Earth has a complex shape and cannot be accurately represented by any geometric figure. In 1873, the German physicist Listing proposed the concept of geoid to characterize the figure of the Earth.

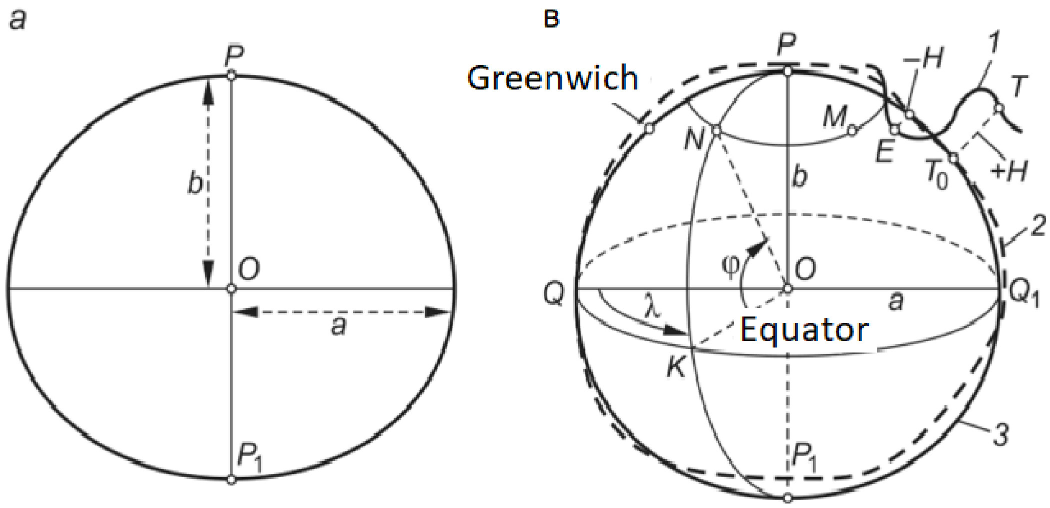
§ 3. Ellipsoid of Krasovsky.

Earth figures. The dimensions and shape of the physical surface of the planet Earth are attributed to one or another of its geometrically correct models, the surface of which is used as the basis for establishing global, regional or private coordinate systems for geodetic work and mapping.

The real surface of the earth's crust is a relief, expressed by combinations of irregularities of various sizes and shapes. The waters of the World Ocean cover more than 71% of the solid surface of the Earth, so its surface served as the basis for creating a physical model of the Earth, representing the figure of our planet. A smooth, convex surface everywhere, formed by the water level of the World Ocean in a state of complete rest and balance, mentally continued under land, is called the *geoid.* The surface of the geoid at each of its points is perpendicular to the direction of gravity (plumb line), i.e. is horizontal everywhere and represents *the main level surface* , relative to which the heights of points on the earth's surface are measured in the accepted system. Due to the fact that in different countries the position of the geoid is determined from the water level in the nearest sea or ocean, different systems of heights are adopted. For example, in Belarus we have adopted the Baltic system of heights, in which the reference surface is the geoid surface passing through the zero of the Kronstadt footstock, which fixes the average water surface level of the Gulf of Finland in the Baltic Sea. Due to the uneven distribution of density in the earth's crust and relief, the geoid surface has global and local waves and does not have a strict geometric description; therefore, it is impossible to solve the problems of calculating and transmitting the coordinates of points on the earth's surface on it. To solve these problems in geodesy, a mathematical model is used - a common earth ellipsoid, represented by an ellipsoid of revolution, compressed at the poles, the axis of rotation of which and the geometric center coincide with the axis of rotation and the center of mass of the Earth for a certain epoch (Fig. 1., a).

Systems of geodetic (geographical) coordinates (latitudes cf , longitudes *X),* referred to the surface of such an ellipsoid, are called *global geocentric.*

In 1940, on the basis of degree measurements made on the territory of the USSR and in a number of other countries, under the scientific guidance of Professor F.N. Krasovsky, the most accurate for that time parameters of the common terrestrial ellipsoid were obtained: the dimensions of its major semi-axis *a* = 6 378 245 m and the polar compression a ~ *(a-b) I a* - 1/298.3. This ellipsoid was



Rice. 1. Earth ellipsoid:

a - ellipsoid of revolution; *b* - geographical coordinates (surfaces under study: *1* - Earth; *2* - geoid; *3* - earth ellipsoid)

oriented relative to the geoid under the condition of minimal deviations of their surfaces only on the territory of the USSR. The ellipsoid with the indicated parameters and orientation in the body of the Earth was given the name "Krasovsky's reference ellipsoid". This term indicates that the given ellipsoid is the most suitable to the geoid not on the entire surface of the Earth, but only on its part. The centers of geodetic points of triangulation polygons of the 1st class of the State Geodetic Network of the USSR were projected onto its surface, their coordinates were calculated using the geometry and parameters of this ellipsoid. Thus *,* ***the SK-42 coordinate system*** was fixed on the territory of the USSR , which is still used on the territory of the Republic of Belarus. After joint mathematical processing of a continuous astronomical- geodesic network of the 1st-2nd classes on the territory of the USSR, completed by the 1990s, as well as using satellite determination data accumulated by that time, a reference was fixed on the surface of the Krasovsky ellipsoid. - Rents system of geodetic coordinates SK-95. To date, with the rapid development of satellite methods in geodesy, modern general earth geocentric coordinate systems have been obtained. Examples of such systems are WGS-84 (USA), PZ-90 (Russia).

Height coordinate ***H*** point ***T*** of the earth's surface in engineering and geodetic works is determined along the plumb line 7Uo relative to the surface of the geoid (Fig. 1.,b).

In many practical mine surveying and geodetic calculations, the common earth ellipsoid and reference ellipsoid are replaced by their simpler model - the globe of radius R = 6371 km (the volume of the globe is equal to the volume of the earth's ellipsoid). The length of the equator L on the ellipsoid F.N. Krasovsky is equal to *2pa*, or 40,075 km, on the globe *2nR*, or 40,030 km ("40,000 km").

§ 4. The principle of depicting the earth's surface on a plane.

A level surface is a surface in which the potential of gravity has the same value at all points. The direction of the normal to the level surface coincides with the direction of gravity, i.e. with a sheer line. There are many such surfaces. *A geoid* is a level surface coinciding with the surface of the oceans and seas in a calm state of water masses and mentally continued under the continents.

Due to the uneven distribution of masses inside the Earth, the surface of the geoid is very complex, and it is not possible to establish its shape and dimensions. Since the shape of the geoid is largely determined by the rotation of the Earth at a constant speed around its axis, when solving practical problems, the surface of the geoid is replaced by the surface of an ellipsoid of revolution.

The parameters of the general terrestrial ellipsoid, determined under the guidance of the Soviet geodesist Feodosy Nikolaevich Krasovsky in 1940, are as follows:

a - semi-major axis, 6 378 245 m;

b – semi- minor axis, 6 356 863 m;

compression α = (ab)/a = 1/298.3.

When deriving the parameters, materials of degree measurements from the USSR, Western Europe and the USA were used. Gravimetric survey materials available by that time were used. The compression determined from satellite observations is very close in value to that obtained by F.N. Krasovsky. In 1980, the International Astronomical Union adopted new values for the ellipsoid: a = 6,378,137 m; α = 1/298.25722.

In order for the earth's ellipsoid to come closer to the geoid, it must be appropriately positioned in the body of the Earth, or, in other words, oriented.

An ellipsoid of revolution with certain parameters, oriented in the body of the Earth in such a way that the deviations of its surface from the geoid for a given territory are minimal, is called a reference ellipsoid. By Decree of the Council of Ministers of the USSR No. 760 of April 7, 1946, the Krasovsky ellipsoid was adopted as a reference ellipsoid for geodetic and cartographic work in the USSR. The center of the round hall of the Pulkovo observatory with the established latitude, longitude, and azimuth was taken as the starting point.

Ellipsoid parameter values obtained by other authors:

1841, Bessel (Germany): a = 6,377,397 m; b = 6 356 079 m; α = 1/299.2;

1880 Clark (Great Britain): a = 6,378,249 m; b = 6 356 515 m; α = 1/293.5;

1909, Hayford (USA): a = 6,378,388 m; b = 6 354 912 m; α = 1/297.0.

On fig. 3 shows the relative position of the ellipsoid, geoid and the physical surface of the Earth. The angle ξ between the normal to the ellipsoid and the direction of gravity is called *the deviation of the plumb line* . For the Krasovsky ellipsoid, on average, ξ = 3 - 4″, only in some areas ξ = 1′. The deviations of the ellipsoid from the geoid do not exceed 100–150 m in height.

Frames of reference are general-earth and reference. General terrestrial geodetic reference systems include the parameters of the terrestrial ellipsoid, the gravitational field of the Earth and the Greenwich geocentric rectangular coordinate system.

The most important parameters of the Earth are:

$f \cdot M_3$ - the product of the gravitational constant by the mass; ω_3 - angular velocity of rotation; a - semi-major axis (equatorial radius) and α - compression of the earth's ellipsoid (compression α = (a - b)/ a

, where b is the minor semi-axis of the ellipsoid); C_o - the speed of propagation of electromagnetic oscillations in vacuum.

Table 1 shows the physical parameters of the Earth PZ-90.

Physical parameters of the Earth PZ-90 table 1

$f M_3$	$398600.44 \cdot 10^9$ m ^{3}/ s ^{2}
ω_3	$7292115 \cdot 10^{-11}$ rad _
C_o	299792458 m/ s

The geometric parameters of the ellipsoids currently in use are shown in Table 2.

Table 2

Name of the ellipsoid	Half shaft a, m	Compression α
SK - 42, SK - 95	6 378 245	1/298.3
PZ - 90	6 378 136	1/298.257839303
WGS -84	6 378 137	1/298.257223563
GRS -80	6 378 137	1/298.257222101

By the Decree of the Government of the Russian Federation of July 28, 2000, for geodetic and cartographic work, from July 1, 2002, a unified coordinate system SK - 95 is established, for orbital flights and solving navigation problems - geocentric coordinate system PZ - 90. Until the transition is completed, SK - 42 is used (Krasovsky ellipsoid). The SK-95 coordinate system was built as a result of joint processing of 164 thousand points of the Astronomo - Geodetic network, 134 points of the Doppler and 26 points of the space geodetic networks. It is built on the Krasovsky ellipsoid, the axes of which, unlike SK-42, are oriented parallel to the corresponding coordinate axes of PZ-90.

§ 5. Heights of points on the earth's surface.

Previously, two systems of heights were considered: geodetic height and orthometric height. The discrepancy between these heights is due to the retreat of the ellipsoid from the geoid.

The height differences of points on the earth's surface, obtained from leveling, determine the difference in potentials of gravity between these points. If the value of the potential at the initial point $W_{o \text{ is known}}$, then, based on the results of leveling, it is easy to calculate the value of the potentials of gravity at the corresponding points of the Earth using the formula

$$W_B \backslash u003d\, W_o - \int_O^{in} g\, dh \text{ , (1)}$$

where g is the value of gravity.

Knowing the heights is necessary for depicting the relief of the earth's surface, as well as for the transition from the values measured on this surface (angles, lines) to the values corresponding to them on the surface of the ellipsoid.

From direct measurements, the differences in the heights of points on the Earth are obtained. To calculate the heights, you need to know the height of the point taken as the initial one. It is assumed that the height of the starting point is known. In the USSR, the zero of the Kronstadt footstock (Baltic system of heights) is taken as the beginning of the calculation of heights.

The geodetic height H can be considered as the sum of two terms: the distance from the reference ellipsoid to the surface of the geoid (or quasi-geoid) and the distance from this surface to the corresponding point on the Earth's surface. These terms are located along the normal to the surface of the reference ellipsoid. From Fig. 6, the geodetic height of the point *M* will be equal to

H_{M} \u003d $H^{g}_{m} + \xi^{m}_{1}$; (2)

$H_{M} = H^{v}_{m} + \xi^{m}$, (3)

where H^{g}_{m} is the orthometric height, ξ^{m}_{1} is the height of the point M_{1} above the surface of the ellipsoid.

Molodensky's research showed that without involving hypotheses about the internal structure of the Earth, both terms of the expression cannot be calculated. But it is possible to calculate exactly both terms of the expression, in which H^{v}_{m} is the normal height, ξ^{m} is the height anomaly or the height of the quasi-geoid above the surface of the reference ellipsoid. To calculate geodetic heights *H* in the USSR, a formula is currently used that provides for the use of a system of normal heights H^{v} and height anomalies ξ.

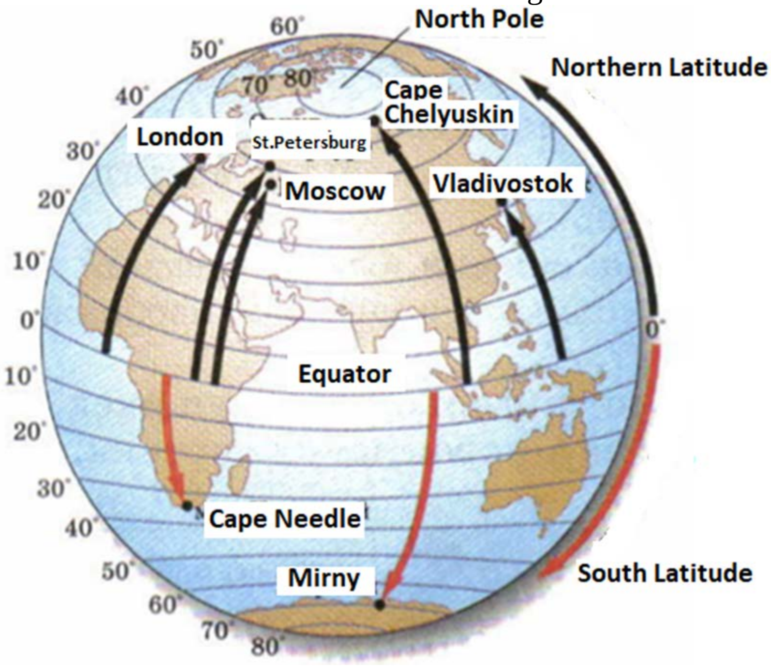
The separation of two terms in the height *H* is connected with the practical need to read heights from sea level. In the system of orthometric heights, the sea level surface is the surface of the geoid, and in the system of normal heights, the auxiliary surface of the quasi-geoid is taken as the sea surface.

The exact height H^{v} is determined by the method of geometric leveling - a method in which, using a horizontal sighting beam, first determine the difference in heights of two neighboring points, and then, using the known height of one of them, find the height of the second point. The influence of the non-parallelism of the level surfaces of the surface of the quasigeoid is taken into account using gravimetric data (in principle, height anomalies can be calculated using the Bruns formula: $\xi = T / \nu$, in which *T* is the perturbing potential, ν is the value of the normal gravity for a point on the surface of the ellipsoid. For more details, see P. S. Zakatov Course of higher geodesy - M.: Nedra, 1976). Published catalogs give normal heights H^{v} benchmarks and centers of triangulation.

Chapter 2
The concept of the coordinate system used in geodesy

§ 6. Geographic coordinate system. Meridians and parallels. Geographic latitude and longitude.

Geographic latitude is determined using parallels. Latitude can be north (those parallels that are north of the equator) and south (those parallels that lie south of the equator). The value of latitudes is measured in degrees and minutes. Geographic latitude can have values from 0 to 90 degrees.



Rice. 2. Determination of latitudes

Geographic latitude - the length of the arc in degrees from the equator to a given point.

To determine the latitude of an object, you need to find the parallel on which this object is located.

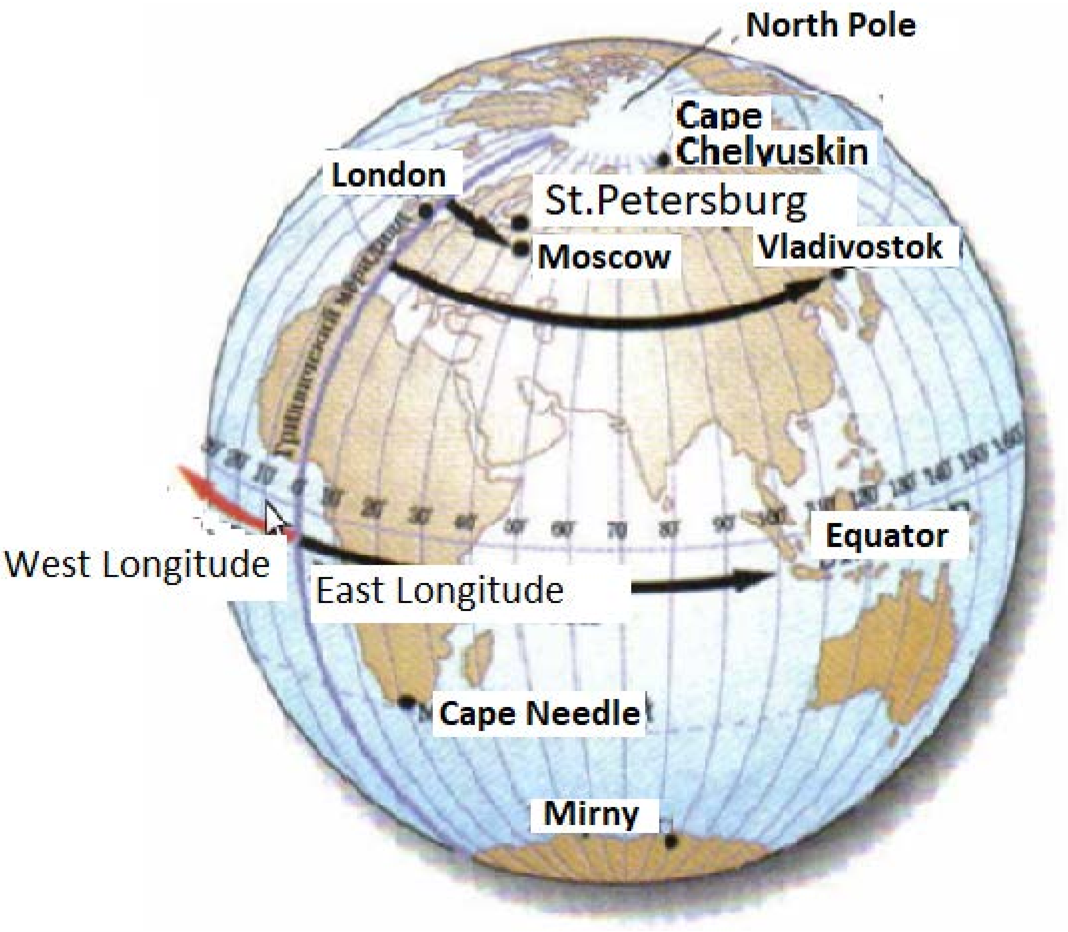
For example, the latitude of Moscow is 55 degrees and 45 minutes north latitude, it is written as follows: Moscow 55 ° 45 'N; the latitude of New York is 40°43'N; Sydney - 33°52'S

Geographic longitude

Geographic longitude is determined by meridians. Longitude can be western (from 0 meridian west to 180 meridian) and eastern (from 0 meridian east to 180 meridian). Longitudes are measured in degrees and minutes. Geographic longitude can have values from 0 to 180 degrees.

Geographic longitude - the length of the equatorial arc in degrees from the initial meridian (0 degrees) to the meridian of the given point.

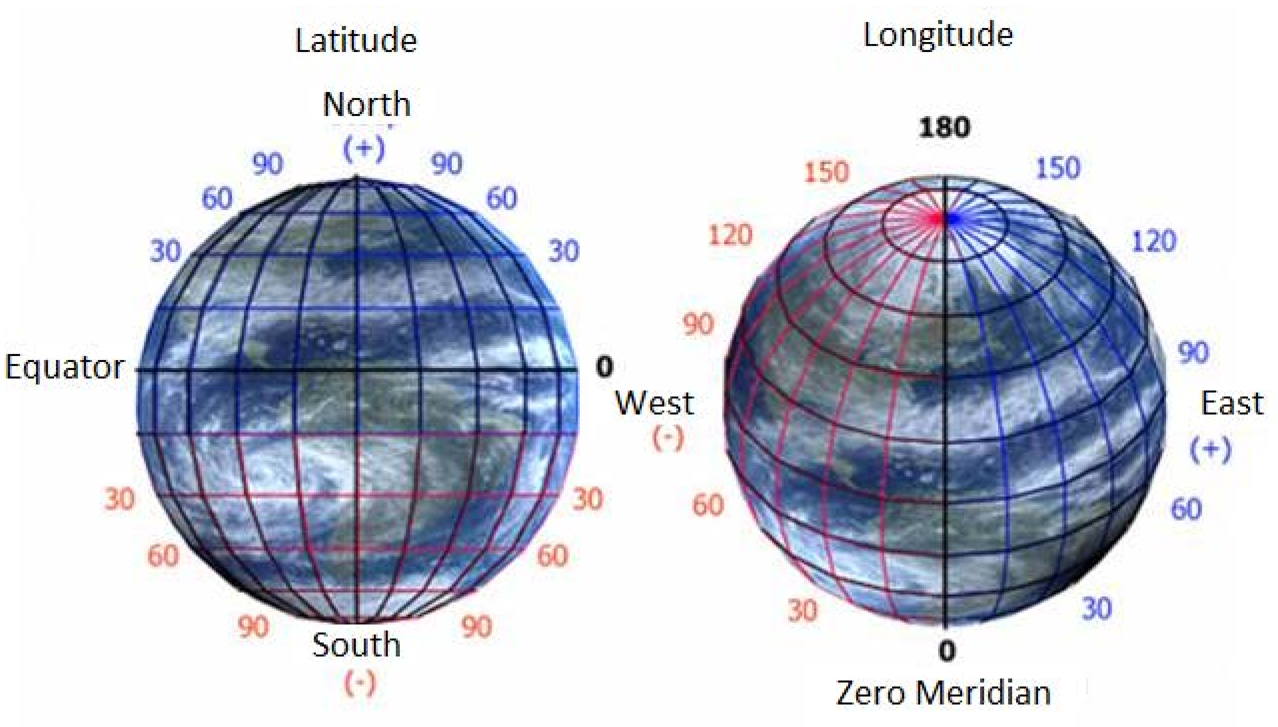
The prime meridian is the Greenwich meridian (0 degrees).



Rice. 3. Determination of longitudes

To determine longitude, you need to find the meridian on which the given object is located.

For example, the longitude of Moscow is 37 degrees and 37 minutes east longitude, it is written as follows: 37 ° 37 'E; longitude of Mexico City – 99°08'W



Rice. 4. Geographic latitude and geographic longitude

Geographical coordinates

To accurately determine the location of an object on the surface of the Earth, you need to know its geographic latitude and geographic longitude.

Geographic coordinates are quantities that determine the position of a point on the earth's surface using latitudes and longitudes.

For example, Moscow has the following geographic coordinates: 55°45'N. and 37°37'E The city of Beijing has the following coordinates: 39°56′ N 116°24′ E The latitude value is written first.

Sometimes you need to find an object by already given coordinates, for this you must first assume in which hemispheres this object is located.

§ 7. System of plane rectangular coordinates. Gauss-Kruger projection. Distribution of six-degree zones. Zonal system of rectangular coordinates. Reshaped ordinates. coordinate grid.

Geodetic coordinate system

For the main surface on which the position of the points of the Earth is determined, the surface of the reference ellipsoid is taken. The coordinate planes, relative to which the coordinates of the points are determined, are the plane of the equator of the earth's ellipsoid and the plane of the initial meridian (Fig. 5).

Geodetic latitude B (**f)** - the angle formed by the normal to the surface of the ellipsoid at a given point and the plane of the equator. The latitudes are counted from the equator from 0 to 90 °, in the northern hemisphere - with a plus sign, in the southern - with a minus sign.

Geodetic longitude L (l) is the dihedral angle between the plane of the initial (zero) meridian and the plane of the meridian of a given point.

The geodetic height of a point H is the distance along the normal from the surface of the ellipsoid to the given point.

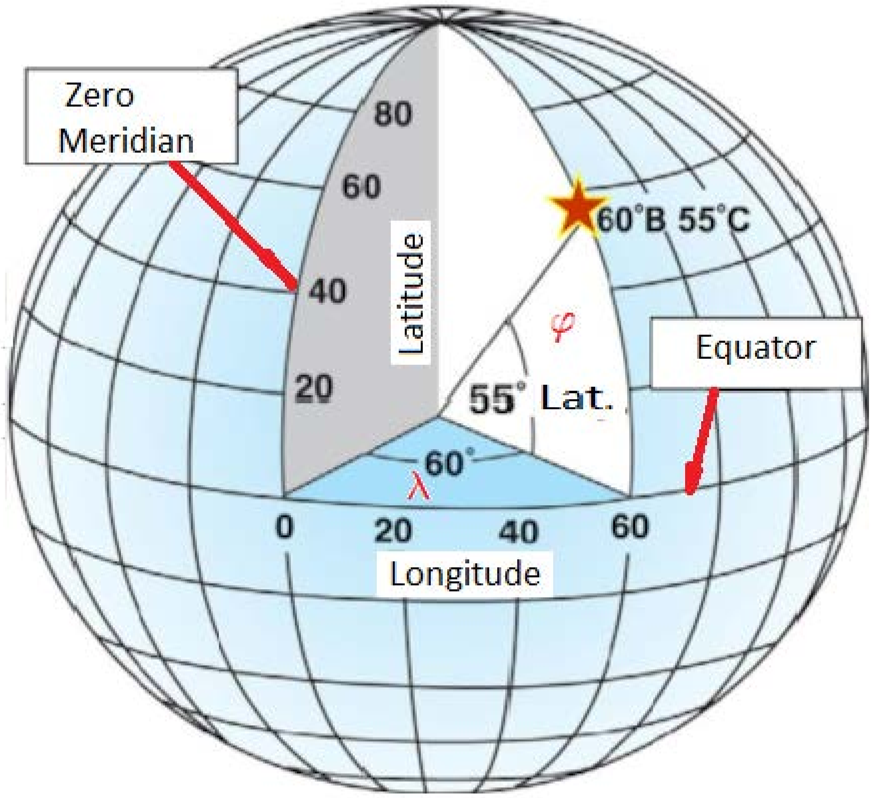


Figure 5. Geodetic latitude and longitude.

Astronomical coordinate system

In contrast to the geodetic coordinate system, in the astronomical system they are determined relative to plumb lines at points on the earth's surface.

Astronomical latitude φ is the angle formed by a plumb line at a given point with the plane of the equator (Fig. 5).

Astronomical longitude λ is the dihedral angle between the plane of the initial astronomical meridian and the plane of the astronomical meridian of a given point.

In this system, the third coordinate is the *orthometric height* H_g - the height of the point above the surface of the geoid.

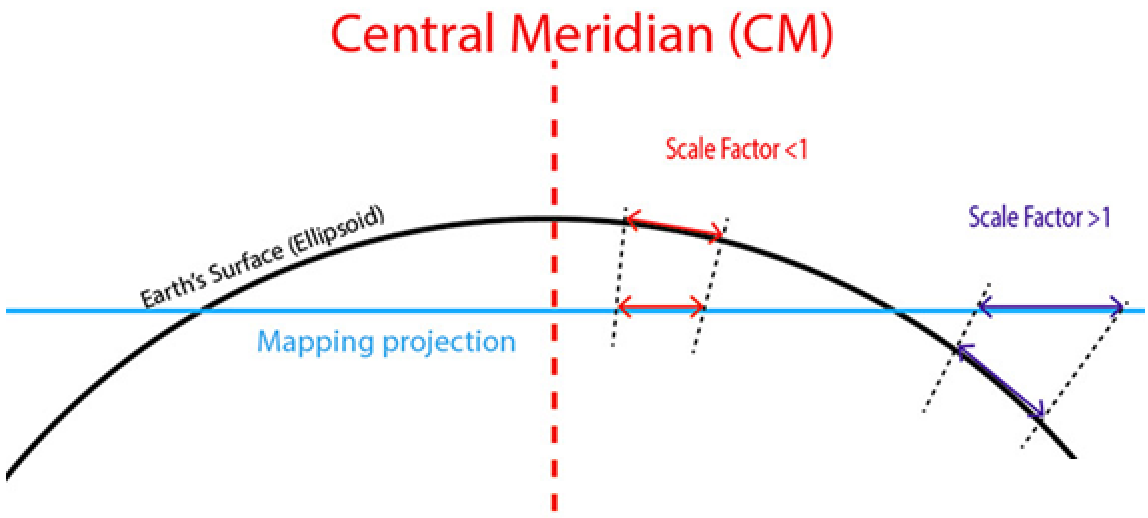
In small-scale mapping, the distinction between astronomical and geodetic coordinates is usually neglected and latitudes and longitudes are used as coordinates of a general system of geographical coordinates.

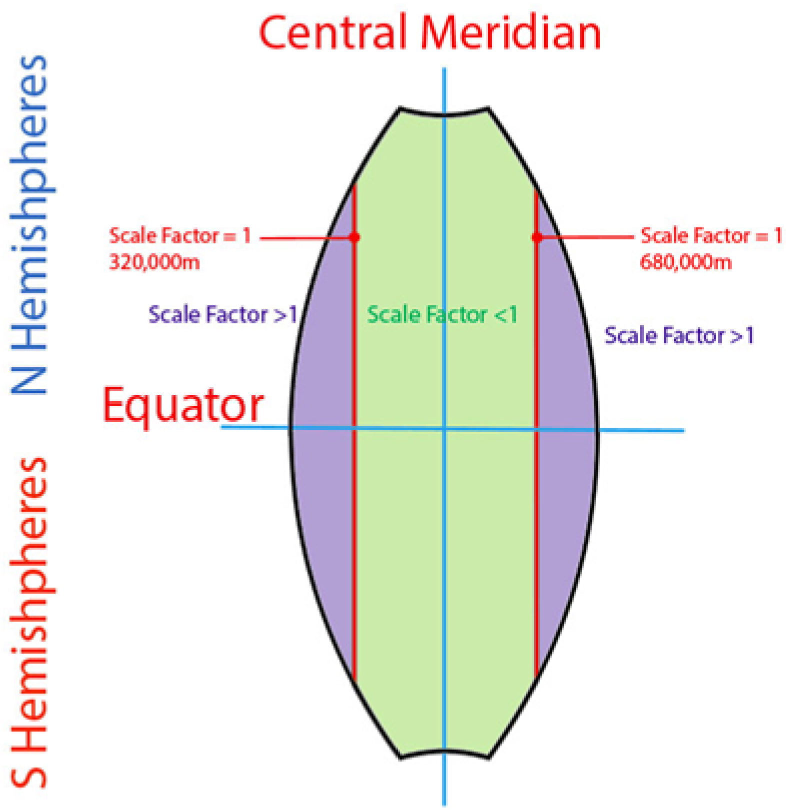
Rectangular coordinate system

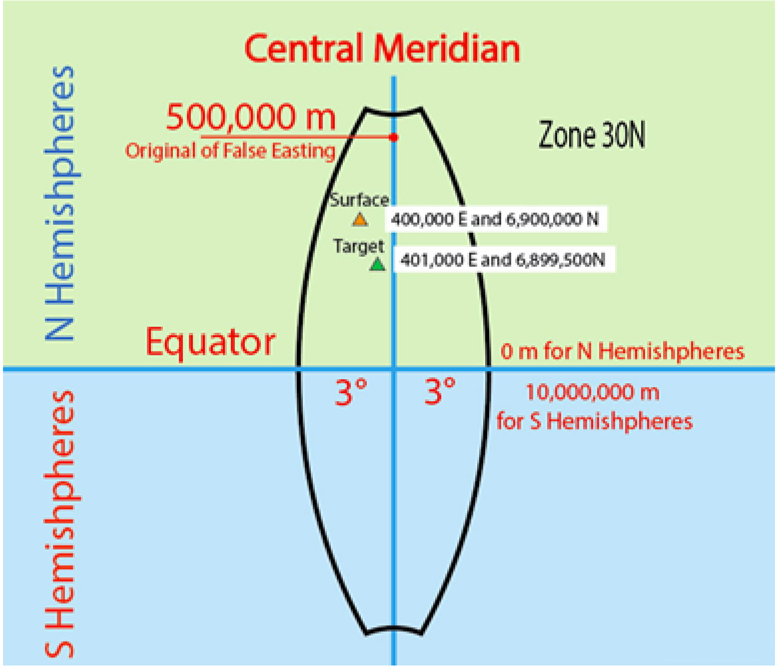
In the spatial system of rectangular coordinates X , Y , Z with the origin at the center of the earth's ellipsoid, the Z axis is located along the polar axis, the X , Y axes are in the equatorial plane. The X - axis is located in the section of the prime meridian, the Y -axis is perpendicular to it. The system is used to determine the coordinates of spacecraft in near-Earth space.

In a system of flat rectangular coordinates, the abscissa axis X is usually aligned with the midday line (meridian) of the point O , called the origin. The north direction is taken as the positive direction of this axis. For the positive direction of the Y - axis , take the direction to the east. Quarters are numbered clockwise. The first quarter is northeast. Sometimes the abscissa axis X is not aligned with the noon line of the point O

, but this line is given another, more convenient direction for work. Such a system is called *a conditional coordinate system* .







Rice. 6. System of plane rectangular coordinates

Gauss–Kruger projection

In the Russian Federation, for the preparation of topographic maps at scales of 1:1,000,000 and larger, the Gauss-Kruger projection is used (equiangular, transverse , cylindrical).

In 1825, the work of Gauss was published, who first solved the general problem of depicting one surface on another while preserving the similarity in infinitesimal parts. A special case of this general problem is a cartographic problem - the image of the surface of an ellipsoid of revolution on a plane. In 1912, Krueger developed working formulas for this projection. Therefore, such a projection is now called the Gauss–Kruger projection.

In the Gauss-Kruger projection, the surface of the earth's ellipsoid is projected onto a plane in parts - zones. The zone occupies 6° in longitude (sometimes zones occupying 3° in longitude are used). The zones are numbered from the zero meridian to the east (Fig. 15). Each zone is individually projected onto a plane in such a way that the similarity of infinitesimal figures is preserved. The central meridian of the zone and the equator (see Fig. 15) are depicted as mutually prependicular lines forming the *X* and *Y coordinate axes* . the remaining meridians and parallels are shown as curves, and the meridians are symmetrical with respect to the axial meridian, the parallels are symmetrical with respect to the equator. In this projection, the length of the axial meridian is transmitted without distortion, the lengths of the remaining lines are longer compared to their horizontal projections on a level surface. The amount of distortion (elongation) of lines on a plane can be calculated by the formula

$$\Delta S = \frac{y_{cp}^{2}}{2R^2} S \text{ , (4)}$$

In which y_{cf} is the distance from the central meridian of the zone to the midpoint of the line; *S* is the length of the curve on the ellipsoid surface; *R* is the radius of the globe.

As you move away from the axial meridian, line distortions increase and reach 0.14% at the edges of the zone (in three-degree zones, these distortions are approximately 4 times less).

If the number of the column *Q is known* , then the number of the coordinate zone of the Gauss-Kruger projection n is determined from the expression:

$$n = Q - 30. \quad (5)$$

The longitude of the axial meridian of the map sheet at a scale of 1:1,000,000 or the longitude of the axial meridian of the coordinate zone L_o is calculated by the formula

$$L_o = 6° Q - 183° \text{ or } L_o = 6° n - 3. \quad (6)$$

To calculate the rectangular coordinates of the Gauss - Kruger projection by latitude and longitude of a point, there are special tables, for example, "Tables of Gauss - Kruger coordinates for latitudes from 32 to 80 ° through 5 ′ and longitudes from 0 to 3.5 ° through 7.5 ′ and tables of frames and half areas of trapezoids.

On the territory of Russia, all abscissas are positive. In order for the ordinates of the points to be also positive, the ordinate of the origin is taken equal to 500 km. And in order to make it clear which zone we are talking about, the zone number is assigned to the ordinates of the points, for example: y \u003d 7 381 252 m - the seventh zone, the point is located west of the axial meridian at 118 748 m.

Ordinates of points that are 500 km long and have a zone number are called transformed ordinates. To facilitate the determination of the rectangular coordinates of points on the maps, parallel to the *X* and *Y axes* , grid lines of rectangular coordinates are drawn.

When depicting the earth's surface in the Gauss-Kruger projection, gaps form between adjacent zones, so certain inconveniences arise when working with maps at the boundaries of adjacent zones. These inconveniences are eliminated by the introduction of overlap strips 2° wide along the western and eastern boundaries of each zone. On all sheets of topographic maps located within these bands, the outputs of the coordinate grid of their zone are given and, in addition, the outputs of the coordinate grid of the neighboring zone are plotted.

Gauss-Kruger Plane Rectangular Coordinate System

This coordinate system is used for a large -scale image of significant parts of the earth's surface on a plane, and therefore, for solving most problems related to the design of building complexes.

The surface is divided by meridians into latitude zones of 3 or 6 degrees longitude. The globe is inscribed in a cylinder so that the plane of the equator is aligned with the axis of the cylinder. Each zone from the center of the Earth is projected onto the side surface of the cylinder. After designing, the side surface of the cylinder is turned into a plane, cutting it along the generators passing through the earth's poles. In the resulting image, the average meridians of the zones and the equator are straight lines, the rest of the meridians and parallels are curves.

The coordinate system in each zone is the same. For the territory of Russia located in the northern hemisphere, the abscissas are always positive. In order for the ordinates to be always positive, the origin is shifted to the west by 500 km. In this case, all points to the east and west of the central meridian will have positive ordinates. Such ordinates are called *transformed.*

Gauss-Kruger Zonal Coordinate System.

This system is based on the transverse-cylindrical conformal Gauss-Kruger projection (named after the German scientists who proposed it). In this projection, the surface of the earth's ellipsoid is divided by meridians into six-degree zones and numbered from the 1st to the 60th from the Greenwich meridian to the east (Fig. 6). The middle meridian of the hexagonal zone is called the axial meridian.

It is combined with the inner surface of the cylinder and taken as the x-axis. To avoid the negative value of the ordinates (y), the ordinate of the axial meridian is taken not as zero, but as 500 km, i.e. move west for 500 km. Before the ordinate indicate the number of the zone.

For example, the record of coordinates X Mn=6350 km, YMn=5500 km indicates that the point is located in the 5th zone on the axial meridian (λMn=27? NL, φMn=54? VD). For approximate calculations, when moving from geographic to rectangular zonal coordinates, it is believed that 1 ^{o} corresponds to 111 km (40,000 km / 360 ^{o}) .

§ 8. Influence of curvature of the earth on the measured distances and heights of points.

The physical surface of the Earth, on which geodetic measurements are performed, deviates differently from the reference ellipsoid at different points, therefore, in higher geodesy, when mathematically processing the measurement results, they are referred to the surface of the reference ellipsoid. The distance *S* between points *A* and *B* (Fig. 7, a) of the Earth's surface when projecting onto the surface of the reference ellipsoid will be equal to S_o. When solving many problems in topography and cartography, the surface of the Earth is taken as the surface of a sphere, and in some cases even as a plane.

Let us determine the dimensions of the area on which the level surface can be considered a plane. On fig. 7, b shows a tangent at a point *And* to the arc *CAB* of radius *R* . The distance along the arc *AB* is equal to *S* , the distance along the tangent is equal to *t* . The difference between the distance along the tangent and along the arc *Δ S \ u003d t - s* .

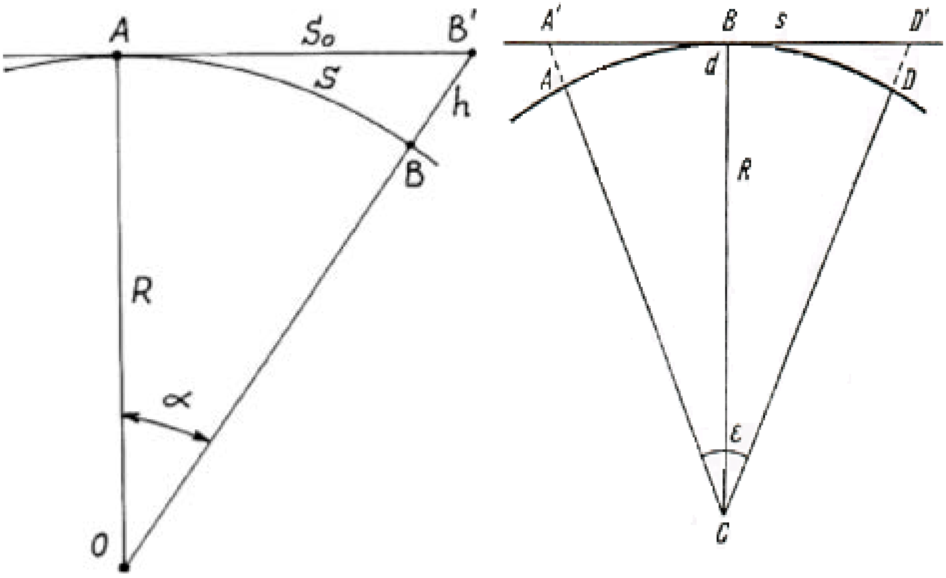


Fig. 7 a, b - The influence of the curvature of the Earth and its definition

From right triangle OAB_1: $t = R\, tg\alpha$. On the other hand, $S = R\alpha$, where the angle α is expressed in radians, then

$$\Delta S = R\,(tg\alpha - \alpha). \quad (7)$$

Expanding *tg α into a series and restricting ourselves to the* first two terms of the expansion due to the smallness of the angle *α , we obtain*

$$tg\alpha = \alpha + \frac{1}{3}\alpha^3. \text{ (eight)}$$

After substituting the value of *tg α* into the previous formula, we obtain

$$\Delta S = R\frac{\alpha^3}{3}, (9)$$

But $\alpha = \frac{S}{R}$, Consequently,

$$\Delta S = \frac{S^3}{3R^2} \approx \frac{t^3}{3R^2}. \text{ (ten)}$$

Below are the values of *ΔS* and the relative errors of *ΔS / S* at different values of *t* (R = 6371 km) .

t , km	10	50	100
Δ S , cm	0.82	26	820
Δ *S/S*	1/1 218 000	1/195 000	1/12 200

At present, the limiting accuracy of line measurements is characterized by a relative error of 1/1,000,000. The surface of the earth can be considered flat.

When determining the heights, the replacement of the *CAB level surface* (see Fig. 7, b) with a tangent *t* leads to an error *Δ h* in the height of the point being determined.

From a right triangle OAB_1:

$$t^2 = (R + \Delta h)^2 - R^2; \qquad \text{(eleven)}$$

$$t^2 = R^2 + 2R\Delta h - R^2; \text{ (12)}$$

$$t^2 = 2R\Delta h + \Delta h^2; \qquad \text{(13)}$$

$$t^2 = \Delta h(2R + \Delta h); \text{ (fourteen)}$$

$$\Delta h = \frac{t^2}{2R + \Delta h}. \qquad \text{(fifteen)}$$

Because *Δh is* small compared to *R* and *t* ,

$$\Delta h = \frac{t^2}{2R}. \text{ (16)}$$

For example, at a distance *t* = 1000 m, *Δ h* = 7.8 cm is obtained. The accuracy of determining heights in topographic measurements is about 5 cm, so the effect of the Earth's curvature must be taken into account.

Chapter 4
Basic survey drawings

§ 12. The concept of a plan and a map.

Until recently, a map was defined as a reduced image of the earth's surface on a plane. But such a definition is imprecise and incomplete. The map is distinguished from other images of the earth's surface by the following features: a mathematically defined construction, the use of special sign systems (cartographic symbols), the selection and generalization of the depicted phenomena, as well as a systematic display of reality. The mathematically defined construction of maps consists in establishing a strict mathematical relationship between the geographical coordinates of points on the earth's surface and rectangular ones - the same points on the plane. This process includes two actions - the projection of the physical surface of the Earth, which is characterized by a complex relief, mapped phenomena and objects onto a mathematical surface, which is taken as the surface of a reference ellipsoid, as well as the image of the surface of the ellipsoid in the required scale on the plane. To move from the surface of an ellipsoid to a plane, one or another mathematical method is used, called map projections.

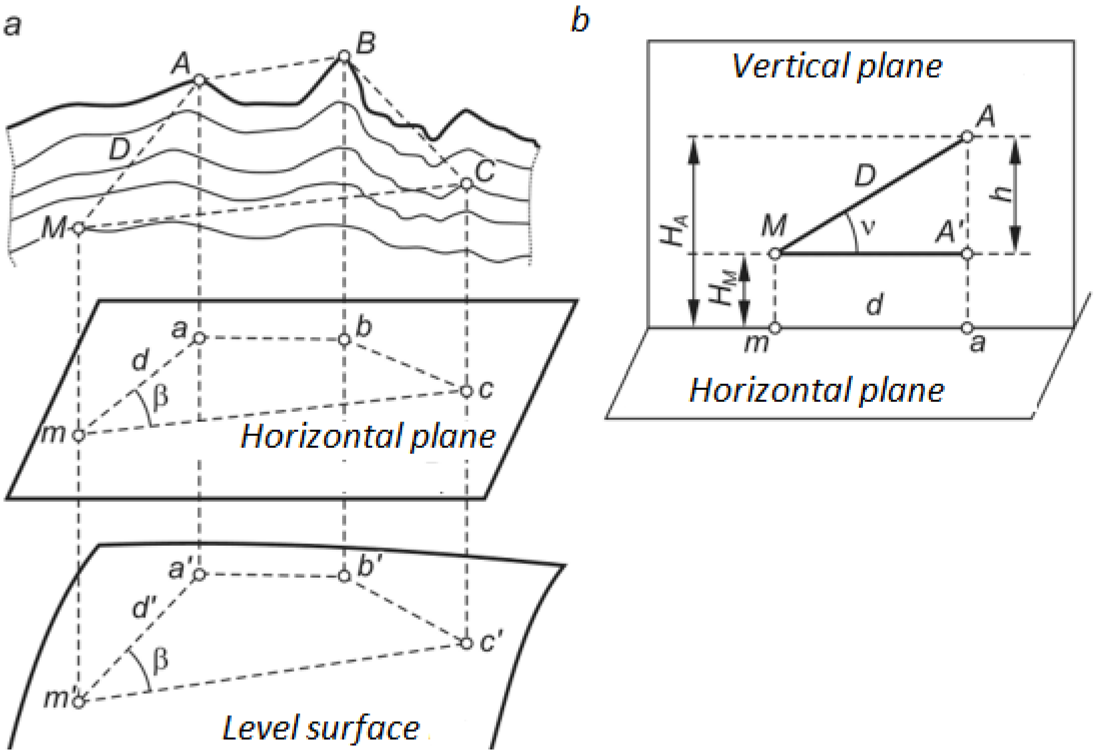
A topographic plan is a reduced and similar cartographic image on the plane of an orthogonal projection of a small area of terrain, within which the curvature of the level surface is not taken into account.

When drawing up a plan, the lines measured on the ground are orthogonally projected onto a horizontal plane. The horizontal projection of the line D - the horizontal distance d (Fig. 8,b) is determined by the formula

$$d = D \cos v, \qquad (17)$$

where *D is* the distance measured between points *A* and *B* on the ground; *v is* the angle of inclination of the terrain.

Otherwise, this method is called the method of horizontal projection and is used when performing geodetic work to display their data on a horizontal plane in the form of numerical values and cartographic drawings. The points of the ABCM contour of the earth's surface (Fig. 8, a) are projected onto the level surface Ru with plumb lines. On a level surface, points a', b', c', m', lines m'a', m'c ', a'b', ..., as well as the contour a'b'c't' represent plumb projections of the corresponding ABCM contour elements. For a limited area on the horizontal plane of the RT, the orthogonal projection is carried out by practically parallel vertical rays: points *a, b, c, t are obtained;* lines *ta, ts, ab, ...,* as well as the contour *abst.* In engineering practice, the horizontal plane P_g is brought closer to the level surface on the territory of the city, industrial enterprise.



Rice. 8 Orthographic projection of the terrain:

a - on a horizontal plane and a level surface;

b - horizontal spacing

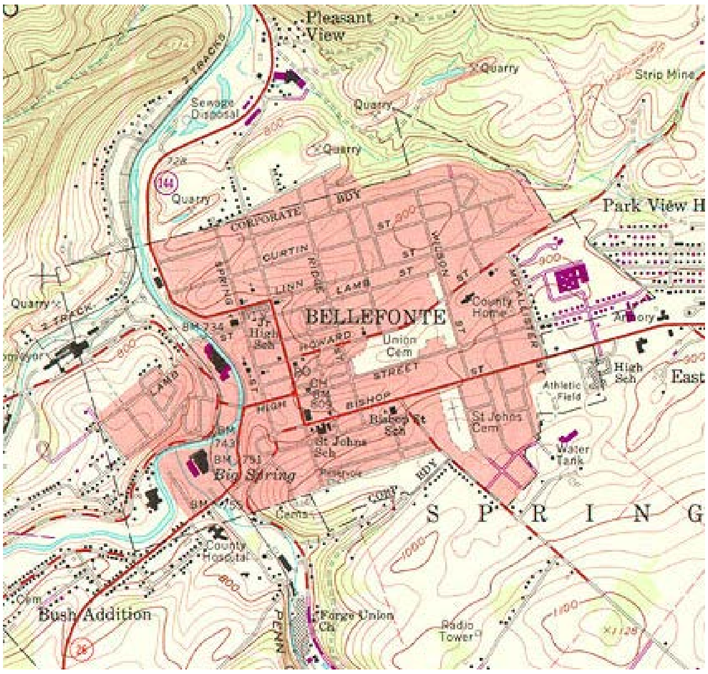
§ 13. Scales.

The scale of the plan is the ratio of the length of the segment on the plan to the length of the horizontal distance of the corresponding segment on the ground. The scale of the plan is constant in all its parts. On maps, a partial scale and a main scale are distinguished.

The partial scale is the ratio of an infinitely small segment *Δ d* on the map (on the plane) to the corresponding segment on the surface of the ellipsoid *ΔS* :

$$\frac{1}{m} = \frac{\Delta d}{\Delta S}. \text{ (eighteen)}$$

The scale of the map is different at its different points: the map usually indicates the only scale value - *the main* (or general) *scale* . On maps covering large areas, there are significant deviations of private scales from the main one. Therefore, on such maps, points or lines of the cartographic grid are indicated, on which the main scale is observed. For example, the scale is 1:2,000,000 at the 45° parallel. The main scale can be used throughout the map without sacrificing accuracy when working with topographic maps (maps at scales of 1:1,000,000 and larger).



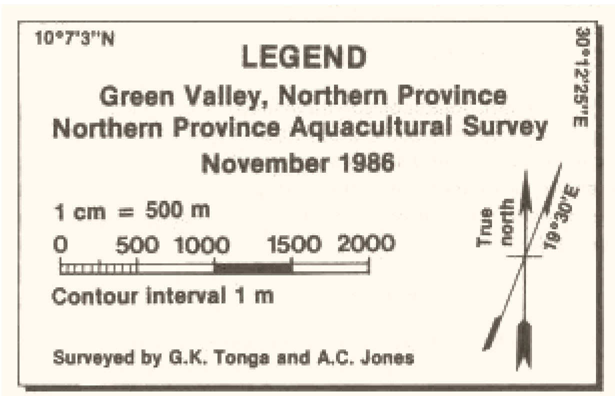


Fig.9. Scale representation on a topographic map sheet.

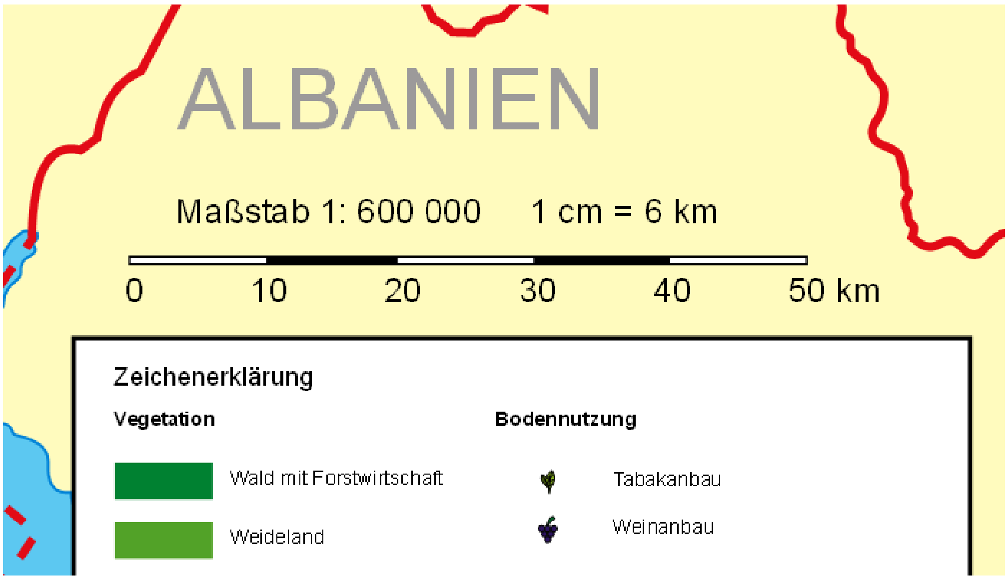


Fig.10. Numerical, named and linear scales.

According to the method of expression, numerical, graphic and named scales are distinguished.

The numerical scale is expressed as a simple fraction 1/ M . The scale denominator is an abstract number showing how many times the horizontal spacing of the terrain lines is reduced when they are displayed on the map. The larger the denominator of the numerical scale M , the smaller the scale, and vice versa, the smaller the denominator M , the larger the scale.

When using a numerical scale, one has to perform calculations that, with a large amount of work, take a significant amount of time. In these cases, use a graphical scale (linear or transverse).

The named scale indicates the length of the line on the ground corresponding to a certain segment on the map (for example, 1 cm 10 km).

Scale Accuracy

The naked eye can distinguish a segment of at least 0.1 mm on the map. The horizontal segment on the ground corresponding to 0.1 mm on the map is called the scale accuracy. The accuracy of scales 1:1,000,000, 1:100,000, and 1:25,000 is 100, 10, and 2.5 m, respectively.

A cartographic projection is a mathematically defined way of depicting the surface of the earth's ellipsoid on a plane, in which each point of the earth's ellipsoid corresponds to a one-to-one point of the plane.

The general formulas for cartographic projections that relate the geodetic coordinates of a point B , L on an ellipsoid with its rectangular coordinates on a plane are as follows:

$$x = f_1(B, L), \quad (19)$$
$$y = f_2(B, L). \quad \text{(twenty)}$$

These formulas allow you to calculate the rectangular coordinates of the displayed point from the geodetic coordinates B and L . The number of projections and, consequently, projection formulas are many. A necessary condition for each projection is a one-to-one correspondence of points with coordinates B , L on the ellipsoid to points with coordinates x , y on the plane and image continuity.

The surface of an ellipsoid (or sphere) cannot be turned onto a plane without deformations (compression or tension), so the requirement for continuity and uniqueness of the image leads to deformation of the ellipsoid surface when it is displayed on a plane. As a result, the scale of the image of the surface on the plane in different parts is different. It is known from the theory of mathematical cartography that an infinitesimal circle on the surface of an ellipsoid is depicted on a plane by an ellipse called *ellipse of distortion.*

The scale of the image depends not only on the position of the point, it varies from point to point depending on the direction. On the map, a main (or general) scale and a private scale are distinguished. The difference between the private scale and the main scale characterizes the length distortion.

Area distortion is characterized by the ratio of the area of the distortion ellipse *dP '* on the map to the area *dP* corresponding infinitesimal circle on the ellipsoid.

$$n = \frac{dP'}{dP}. \quad (21)$$

Angle distortion is the difference between the angle formed by two lines on the ellipsoid and the image of this angle on the map. Angle distortions are different for different directions coming out of a point. As a characteristic of the distortion of the corners at a given point, the largest value of the distortion is indicated.

There are no map projections without distortion. In projections without distortion, the similarity and proportionality of geometric figures on the entire earth's surface would be preserved, which is possible only on the model of the earth's ellipsoid. However, projections are available that are free of angle distortion or area distortion.

Projections are classified according to two criteria: by the nature of the distortions and by the type of coordinate grid. According to the nature of the distortions, conformal, equal-area and arbitrary projections are distinguished.

Conformal projections transmit the angles of geometric shapes without distortion.

Equal-area projections preserve areas (the scale of areas at each point is the same), but greatly violate the similarity of the figures.

There are many projections which are neither equal area nor conformal and which are called *arbitrary.*

In most cases, when creating maps, geographical coordinates are used - latitude φ and longitude λ. In a cartographic projection (on a map), the geographic coordinate system is set by a cartographic grid of parallels and meridians, which is called *the main one* . In addition to the main one in cartography, a normal cartographic grid is used.

In many projections, the main and normal grids coincide, but sometimes some other grid that can be built on the surface of a ball or ellipsoid has a simpler view in the projection. For a ball, for example, a system of spherical coordinates is used, similar to the geographical system, but its pole with geographical coordinates φ_o, λ_o can occupy different positions. The coordinate lines of the normal system, similar to parallels, are called almucantarats, and the lines, similar to meridians, are called *verticals.*

When the pole of the normal system coincides with the geographic pole ($\varphi_o = 90°$), the network of verticals and almucantars merges with the network of meridians and parallels, the main and normal grids coincide, and the projection is called normal. When the pole of the spherical system coincides with the equator (φ_o \u003d 0 °), the verticals and almucantarata do not coincide with the meridians and parallels, and the projection is called transverse. In the case when the pole of the spherical system is between the pole and the equator ($0 < \varphi_o < 90$ °), the verticals and amulcantarates also do not coincide with the meridians and parallels, and the projection is called oblique.

According to the type of meridians and parallels of the normal grid, the projections are azimuthal, conical, cylindrical, pseudoconical, polyconic, circular, etc.

Let's consider examples of some cartographic projections.

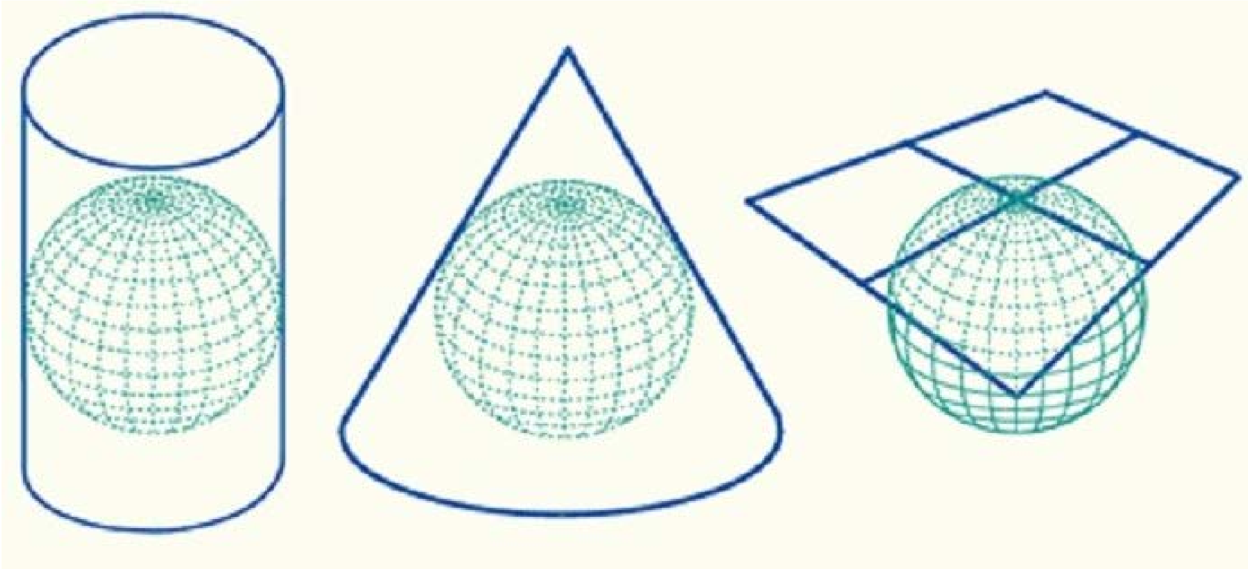


Fig.11. Types of projections: cylindrical , cone and azimuthal

Azimuthal projections

In azimuth projections, almucantarata are depicted as concentric circles, verticals - as straight lines emerging from the center of these circles; the angles between the verticals on the projection are equal to the corresponding angles in nature. According to the position of the center of spherical coordinates, they can be normal, transverse and oblique.

For an example in fig. 11c shows the normal azimuth projection used to image the polar regions of the globe. In it, almucantarats coincide with parallels, and verticals - with meridians.

By the nature of the distortions, azimuth projections can be very diverse.

Conic projections

In conic projections, almucantars of a spherical coordinate system are represented by arcs of concentric circles, and verticals by straight lines converging in a common center of circles; the angles between the verticals are proportional to the corresponding angles in nature, the coefficient of proportionality can take the values $0< k <1$. On fig. 11b, a normal conic projection is shown (in it, the meridians coincide with the verticals, and the parallels coincide with the almucantars).

To clarify the construction of conic projections, one can imagine that the surface of an ellipsoid or ball is depicted on the lateral surface of the cone (tangent or secant), oriented in a certain way relative to the axis of rotation of the ellipsoid or ball. Then the surface of the cone unfolds into a plane.

In oblique and transverse conic projections, meridians and parallels are depicted as complex curves.

Cylindrical projections

Schematically, the construction of such projections can be explained if we imagine that the surface of the ellipsoid (or ball) is projected onto the side surface of the cylinder, tangent or secant, and oriented in a certain way relative to the ellipsoid or ball. The side surface of the cylinder is cut along one of the generators and unfolded into a plane.

When the axis of the cylinder coincides with the axis of the ellipsoid, a normal cylindrical projection is obtained. When these axes intersect at right angles, a transverse axis is obtained . When the axis of the cylinder and the axis of the earth's ellipsoid intersect at an acute or obtuse angle, oblique cylindrical projections are obtained.

On fig. 11a shows a normal cylindrical view. In this projection, the meridians are shown as straight lines, parallel to each other and separated from each other at distances proportional to the difference of the corresponding longitudes, and the parallels are shown as straight lines, perpendicular to the meridians.

At the same time, the earth's surface is divided by meridians and parallels into regular trapezoids, the size of the sides of which is set depending on the scale of the map; a special system is used to designate trapezoids.

The surface of the Earth enclosed in a trapezoid is depicted on a plane using one of the projections (conic, polyconic, etc.).

§ 14. Conventional signs and their classification.

Cartographic conventional signs are graphic symbols used on maps to designate various objects and their characteristics. Briefly they are called conventional signs. They indicate: type of object (well, highway, swamp, etc.); its quantitative and qualitative characteristics (flow rate of the well, type of coverage, width of the carriageway of the highway, passability of the swamp, etc.); spatial position, planned dimensions and shape of the object.

Distinguish off-scale, linear and areal conventional signs. *Off-scale signs* determine the location (point) of an object, and the size of objects cannot be determined from them, for example, springs, geodetic points, and stand-alone trees.

Linear conventional signs depict objects that have a significant length (borders, roads, communication lines, pipelines, etc.). *Areal (scale) symbols* denote objects whose image is similar to the original (they can be used to determine the size and shape of objects).

For topographic maps of each scale, uniform topographic symbols are used. Conventional topographic signs are chosen so that they give a clear and visual representation of the area and, in their outline, would resemble the type and nature of the depicted objects. A good knowledge of conventional signs is necessary in order to be able to represent the depicted area on the map. Examples of conditional topographic signs are shown in fig. 12.

Fig.12. An example of symbols

Symbols that are used on topographic maps and plans are mandatory for all organizations conducting topographic work.

Depending on the scale of the plan or map being created, the corresponding symbols are also used. In our country, the currently valid symbols are:

Symbols for a topographic map at a scale of 1:10000. Moscow: Nedra, 1977.

Symbols for topographic plans at scales 1:5000, 1:2000, 1:1000, 1:500. Moscow: Nedra, 1973.

Symbols, font samples and abbreviations for topographic maps at scales 1:25000, 1:50000, 1:100000. M.: Nedra, 1963.

Conventional signs for ease of use are grouped according to homogeneous features and placed in tables consisting of a serial number, the name of the conditional sign and its image. At the end of the tables are explanations for the application and drawing symbols , as well as an alphabetical index of conventional symbols with their serial numbers, a list of abbreviations of explanatory inscriptions, frame design samples and font samples indicating the name of the font, its size and index according to the "Album of Cartographic fonts" .

§ 15. Representation of relief on plans and maps.

Terrain is a collection of irregularities in the earth's surface. Relief is a very difficult surface to depict. The complexity of the image of the relief is due to the fact that we usually observe it in perspective, while on the map it is depicted orthogonally. For topographic maps, the main method for depicting relief is the method of contour lines (isogypsum).

Horizontal - a closed curved line depicting the locus of points of the earth's surface of the same height. A single horizontal line is not sufficient for depicting relief elements. To transfer the surface, a system of such lines is needed. To represent the geometric essence of contour lines, let's imagine a basin, in the middle of which there is a section of the earth's surface. We mark the coastline of the water's edge, then we will lower the water level in equal steps and mark the obtained closed curves, which are horizontal lines (Fig. 17). The distance between adjacent horizontals along a plumb line *h* is called the *height of the relief section* ; the value of the section height is indicated on each sheet of the map under the linear scale. The distance between horizontals in plan *d* is called the *laying.*

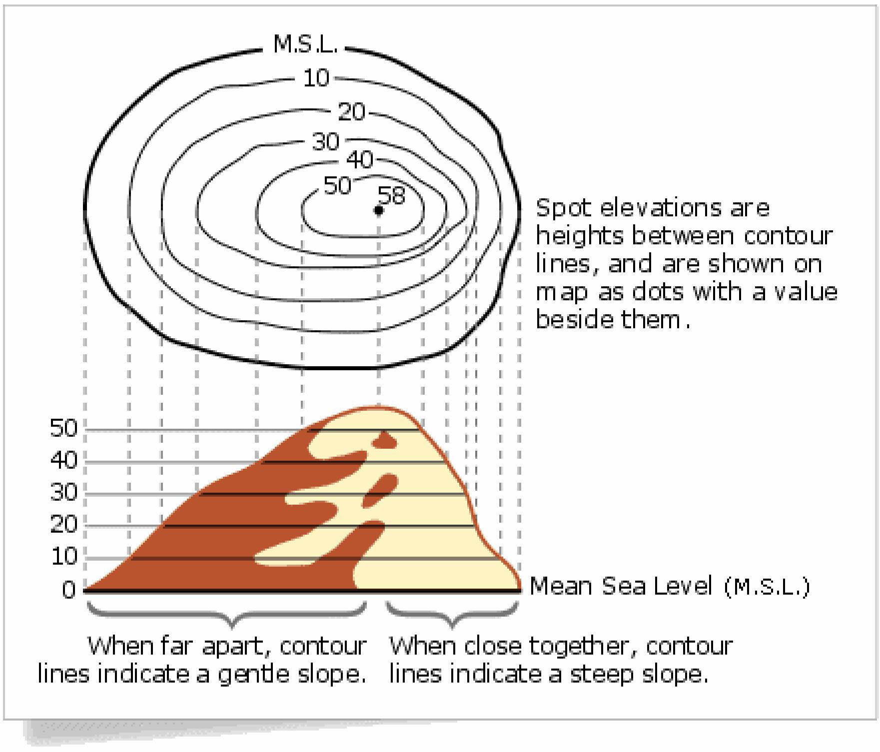


Fig 13. Construction of contour lines

The following properties follow from the definition of contour lines: contour lines are closed curves; horizontal lines cannot intersect; the smaller the distance between contour lines on a map of a given scale, the steeper the slope on the ground.

The choice of the height of the relief section is related to the scale of the map and the nature of the relief. On overview maps, it is important to see large and medium landforms, for the image of which a relatively large sectional height is used. On large-scale maps, small forms should also be shown, therefore, a small sectional height is used to depict them.

It turns out that the smaller the height of the section, the more detailed the relief is depicted. There is a certain limitation in depicting steep slopes. So, in 1 mm it is practically impossible to draw more than four parallel lines so that they are visually perceived separately. If we assume that ν is the limiting angle of the slope of the area represented by contour lines, and *AC* is the minimum horizontal distance between adjacent contour lines (0.25 mm on the map and 0.25 *m* , mm in nature, where *m* is the denominator of the numerical scale of the map), then the value of the minimum section height (in mm) can be determined by the formula

$$h = AC\ tg\nu = 0.25\ m\ tg\nu . \qquad (22)$$

For topographic maps, the heights of the relief section are taken, indicated in Table. 2.

The average relief survey error is 1/3 of the section height. The average height errors of the characteristic points signed on the map should not exceed 75% of the average terrain survey errors, and in mountainous and high-mountainous areas, the number of contour lines must be consistent with the height difference between the slope bends.

Average errors in the position of contour lines in height, relative to the nearest points of the survey justification, on maps of scales 1:50,000 and 1:100,000 of flat-flat areas should not exceed 3 and 6 m, respectively, and of flat, rugged and hilly areas, with prevailing angles of inclination of the surface up to 6° - 4 and 9 m. Maximum errors in height, exceeding twice the values of the average errors, are not allowed on the maps.

Table 3 **Height of relief section on topographic maps**

Territory characteristics	Relief section height (m) for scales				
	1:10 000	1:25 000	1:50 000	1:100,000	1:200,000
Flat plains with terrain slope up to 1°	1.0	2.5	10	20	20
Flat with a slope of terrain from 1 to 2°	1.0 *;	2.0 2.5; 5.0**	10	20	20
Flat crossed and hilly with a slope terrain from 2 to 6°	2.0 (2.5)	2.5***;5.0	10	20	20
Alpine	–	10	20	40	80

* In areas of land reclamation construction

** In forested areas

***In open areas with slopes up to 4°

Errors in the position of contour lines on maps of forested areas should not exceed twice the errors established for maps of the corresponding unforested areas.

On maps of mountainous, high-mountainous regions, as well as areas of sandy deserts, horizontal lines must geographically correctly display landforms, be consistent with the elevation marks signed on the maps and the heights determined at the bends of the slopes.

With the help given in table. 3 sections depict slopes, the steepness of which does not exceed 40 °, with a greater steepness, the horizontals merge.

In some cases, the established section heights do not provide the transfer of the characteristic features of the relief. To overcome these difficulties, additional contour lines are used or different relief sections are used for mountainous and flat areas.

Additional horizontals are drawn in broken lines and are used as necessary to depict details and forms characteristic of a given relief, but not depicted by the main horizontals. If necessary, half-horizontals or quarter-horizontals are drawn on the maps, i.e. horizontal multiples of half or a quarter of the main section. Sometimes the introduction of semi-horizontals or quarter-horizontals is not enough. So, in the Caspian lowland, these horizontals do not always reflect such characteristic relief forms as estuaries (extensive depressions 1-2 m deep). In such cases, auxiliary horizontals of an arbitrary section are introduced with the obligatory signature of their height. Usually, on topographic maps within a single sheet, the height of the section remains constant. This is necessary for the correct transfer of the steepness of the slopes. Changing the height of the section when depicting the relief on a separate sheet would create the impression of an inflection of the slopes on the boundary horizontal.

From Table. 3 it follows that the height of the relief section increases with decreasing map scale. An increase in the section, proportional to a decrease in the scale on small-scale maps, leads to the fact that the relief of the lowlands is not depicted. To avoid this, the so-called contour scale is used, in which the height of the section increases with the height of the terrain. For maps of scales 1:500,000 and 1:1,000,000 in Russia, the horizontal scale given below is used.

Altitude zones, m	150 - 500	500 - 1000 (500 - 2000 for 1:500,000)	above 1000 (above 2000 for 1:500,000)
Section height, m	50	100	200

The scale of contour lines is chosen as a result of studying the relief of the depicted territory. At the same time, terrain profiles in characteristic directions are used. For the correct image of the relief, it is necessary that the height of the section be less than the relative height of the relief forms, the image of which is mandatory, otherwise the map may show a plateau instead of dissected mountain systems. In addition to the purely mathematical rendering of the contours of the relief as lines of equal heights, the contours convey the type of relief with their outlines. Sharp forms of relief correspond to sinuous and angular horizontals, soft forms - round and smooth.

Of the diverse forms of relief in topography, the following characteristic forms are distinguished (Fig. 14).

Mountain, hill, stack . The base of the mountain is called the sole, and its highest point is called the top. The top, which has the shape of a flat, is called a plateau, and the pointed peak is called a peak.

A hollow or depression is a bowl-shaped depression closed on all sides. The lowest part of the basin is called the bottom, its upper part is called the edge.

Ridge - an elongated hill, gradually lowering in one direction. If you draw a line on the map through the points with the greatest curvature of the contour lines depicting the ridge, you will get a watershed line.

A hollow is an elongated depression in the terrain, gradually lowering in one direction. A watercourse line (weir or thalweg) passes through the hollow through the points with the greatest curvature of the horizontals.

A saddle is a depression between two adjacent peaks. In the mountains, passes are usually associated with saddles.

The top of the mountain, the bottom of the basin, the low point of the saddle are called the characteristic points of the relief, and the watershed and thalweg are *the characteristic lines of the relief.*

The outlines of the contour lines depicting a mountain and a hollow, a hollow and a ridge are similar, therefore, to facilitate reading the relief, short lines are drawn on some contour lines in the direction of the slope, called *berghstrich* . The inscriptions of the heights of the contour lines are made in such a way that the bases of the figures are directed towards the lowering of the terrain.

Horizontals are convenient for depicting landforms with relatively smooth changes in height, but they are not suitable for conveying sharp changes in height, for example, cliffs, rocky ridges, cracks, and steep-walled ravines. With the help of contour lines, it is impossible to show microforms of relief and off-scale objects, even if their relative height is greater than the relief section.

To overcome these shortcomings, special conventional symbols are used on topographic maps. So, for example, when depicting relief microforms (hummocky surfaces, marsh mounds, moving sand ridges), the external contour of the microforms is indicated and, if necessary, the quantitative characteristics of these microforms are indicated. When depicting microforms, they strive not only to show the boundaries of the distribution of certain forms, but also reflect the patterns of their formation and distribution, for example, the orientation of ridge sands, dunes, and dunes.

The identification and correct image of relief microforms and their dynamics is one of the tasks of topographic interpretation of aerial and space images. Single landforms that are not expressed on the scale of the map, but which have the meaning of landmarks or are obstacles when moving around the terrain, are shown with off-scale conventional signs . Rocks - remnants, boulders, mounds, caves, ledges, gullies, karst funnels. To depict natural landforms, conventional signs are assigned a brown color, and for forms resulting from human activity, black (quarries, waste heaps, dams, road embankments and excavations, etc.) . Special signs are used to depict dynamic landforms, such as dunes, landslide slopes, and growing ravines.

To determine the heights of terrain points along horizontal lines, it is necessary to know their heights, so the labels for the heights of contour lines are made so that it is easy to find the height of any horizontal line. Finding the heights of contour lines is easier when some of them (for example, the fifth ones) are thickened.

In order to increase the visibility of the image of the relief by contour lines, the method of hypsometric or layer-by-layer coloring (hillshade) is used. When developing a scale for layered coloring, it is important that the steps of the scale (colors and shades) are clearly distinguished from each other, and the colors of all the steps of the scale, regardless of the variety of colors, are harmonious.

Hypsometric coloration scales on maps of scales 1:500,000 and 1:1,000,000 are used depending on the height of mountainous regions, starting from 500 m. when exceeding more than 500 m.

Maps can be classified according to a number of features: scale, territorial coverage, theme, purpose, mathematical basis, etc.

When classifying maps by subject (content), first of all , general geographical and thematic maps are distinguished. In turn, general geographic maps are divided into survey, survey - topographic and topographic.

If we classify maps by scale, then we can distinguish : small-scale (smaller than 1:1,000,000), medium-scale (smaller than 1:200,000) and large-scale (1:200,000 and larger).

Topographic maps have their own scale division: small-scale (1:50,000; 1:100,000; 1:200,000), medium-scale (1:10,000; 1:25,000), large-scale or topographic plans (1:500; 1: 1000; 1:2000; 1:5000).

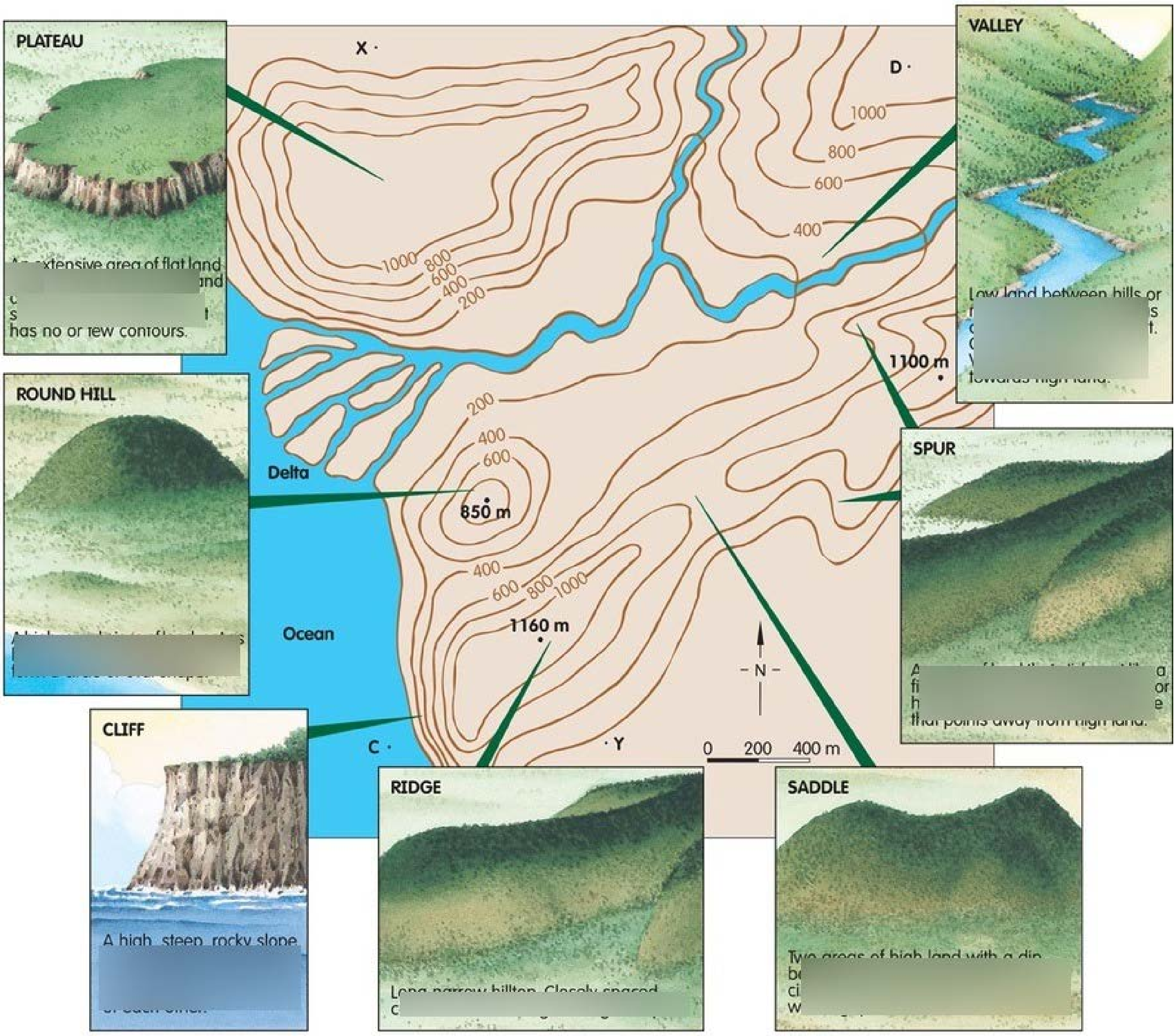


Fig 14. Basic landforms

§ 16. Nomenclature of maps and plans.

Topographic maps of Russia are created in the Gauss-Kruger conformal transverse-cylindrical projection. The height system on these maps is Baltic. Since topographic maps are multi-sheet, a special sheet designation system, the nomenclature, is used to designate each sheet of the map .

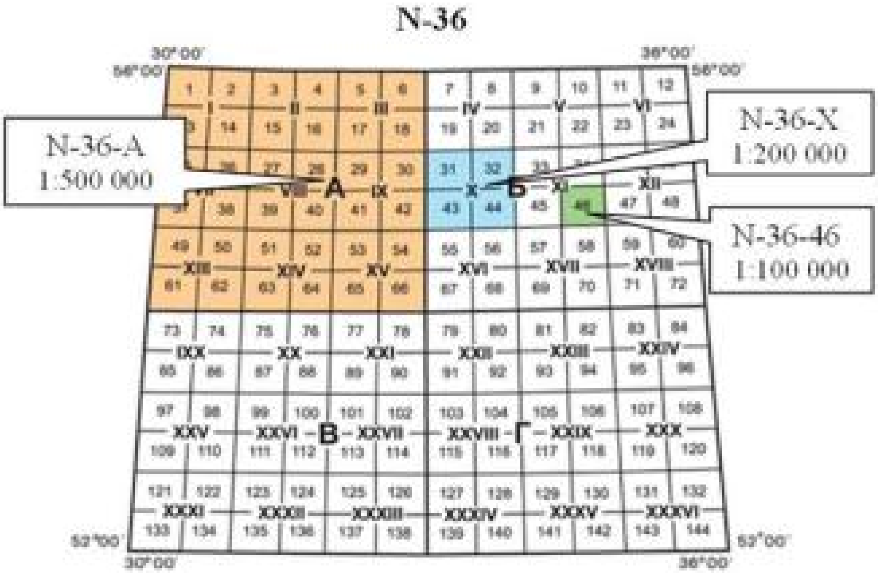
The system of dividing maps into sheets is called layout. For topographic maps and plans, two marking systems are used. So, for maps of small, medium and most large scales, the boundaries of map sheets are the lines of the cartographic grid (lines of meridians and parallels). And for some plans of scales 1:1000 and 1:500, the boundaries of the sheets are the lines of a rectangular coordinate grid (coordinate lines).

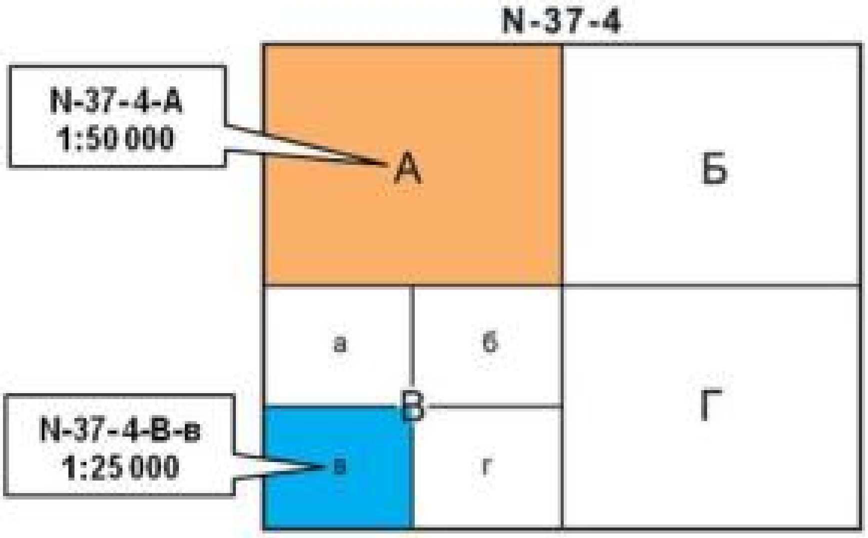
Table 5

Scale	Sheet dimensions		Average leaf area at latitude, 54, km ^{2}	Number of sheets in one card sheet 1:1,000,000	Nomenclature
	By latitude	Longitude			

1:1 000 000	4°	6°	175 104	one	*N* -38
1:500 000	2°	3°	43 776	four	*N* -38- *A*
1:200 000	40′	1°	4864	36	*N* - 38 - XXII
1:100 000	20′	30′	1216	144	*N* - 38 - 133
1:50 000	10′	15′	306	576	*N* -38-133- *A*
1:25 000	5′	7°30′	76	2304	*N* –38–133– *Aa*
1:10,000	2′30 ″	3′45″	19	9216	*N* –38–133– *A*–*a* -2
1:5000	1′15″	1′52.5″	–	–	*N* –38–133–(251)
1:2000	25″	37.5″	–	–	*N* –38–133–(125- *e*)

The nomenclature of Russian maps is based on the international layout of map sheets at a scale of 1:1,000,000. A map sheet of this scale occupies 4° in latitude and 6° in longitude (Table 5). When dividing the surface of the globe (Fig. 15) with parallels from the equator through 4 °, *belts are obtained* , which are indicated by letters of the Latin alphabet from *A* to *V* , starting from the equator north and south. And when dividing by meridians through 6 °, *columns are formed* , which are numbered in Arabic numerals, starting from the meridian with a longitude of 180 ° in the direction from west to east. The nomenclature of a map sheet at a scale of 1:1,000,000 consists of the letter that denotes the belt and the number of the column. On fig. Sheet 15 has the nomenclature *N* - 3 6.





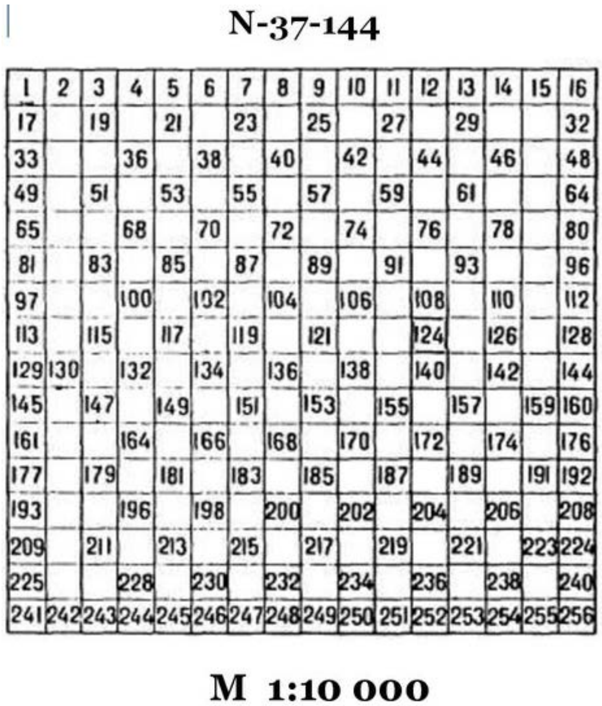


Figure 15 a, b, c. Layout and nomenclature of map sheets

The layout of sheets of a map at a scale of 1:500,000 is carried out by dividing each sheet of a map at a scale of 1:1,000,000 into four parts (Fig. 1 5, a). The nomenclature of a map sheet of this scale consists of the nomenclature of a map sheet at a scale of 1:1,000,000 and the corresponding capital letter of the Russian alphabet from *A* to *G*. _ in fig. 15, and the shading shows a map sheet at a scale of 1:500,000 with the nomenclature *N* - 38.

The layout of the sheets of the map at scales of 1:200,000 and 1:100,000 is carried out by dividing each sheet of the map at a scale of 1:1,000,000 by parallels and meridians, respectively, into 36 and 144 parts. On fig. 15,a - map sheet at a scale of 1 : 200,000 with the nomenclature *N* - 36 - X. On fig. 15, c shows the layout and designation of the sheets of the map at a scale of 1: 100,000, the sheet with the nomenclature *N* -36-46 is shaded.

The nomenclature of the 1:50,000 scale map is obtained by dividing the sheet of the scale map

1:100,000 (Fig. 15, a) into four parts, which are indicated by capital letters of the Russian alphabet. On fig. 15, a is shown in color a map sheet at a scale of 1:50,000 with the nomenclature *N* -37-4-A.

The nomenclature of map sheets at a scale of 1:25,000 is obtained by dividing a map sheet at a scale of 1:50,000 into four parts and designating the parts in lowercase letters of the Russian alphabet *a, b, c, d* , (Fig. 15, *b*).

The nomenclature of map sheets at a scale of 1:10,000 is formed by dividing a map sheet at a scale of 1:25,000 into four parts and designating the resulting parts with Arabic numerals 1, 2, 3, 4. in fig. 15, *b* , hatching shows a sheet of a map at a scale of 1:25,000 with the nomenclature *N* -37-4 *-B-c* and a sheet of a map at a scale of 1:10000 with the nomenclature *N* -37-144 *-D-a -3.*

For drawing at a scale of 1:5000, a sheet of a map at a scale of 1:100,000 is divided into 256 parts, which are indicated by Arabic numerals in brackets. And for drawing a map at a scale of 1:2000, a sheet of a map at a scale of 1:5000 is divided into 9 parts and denoted by lowercase letters of the Russian alphabet from *a* to *u* (Fig. 16 a , b).

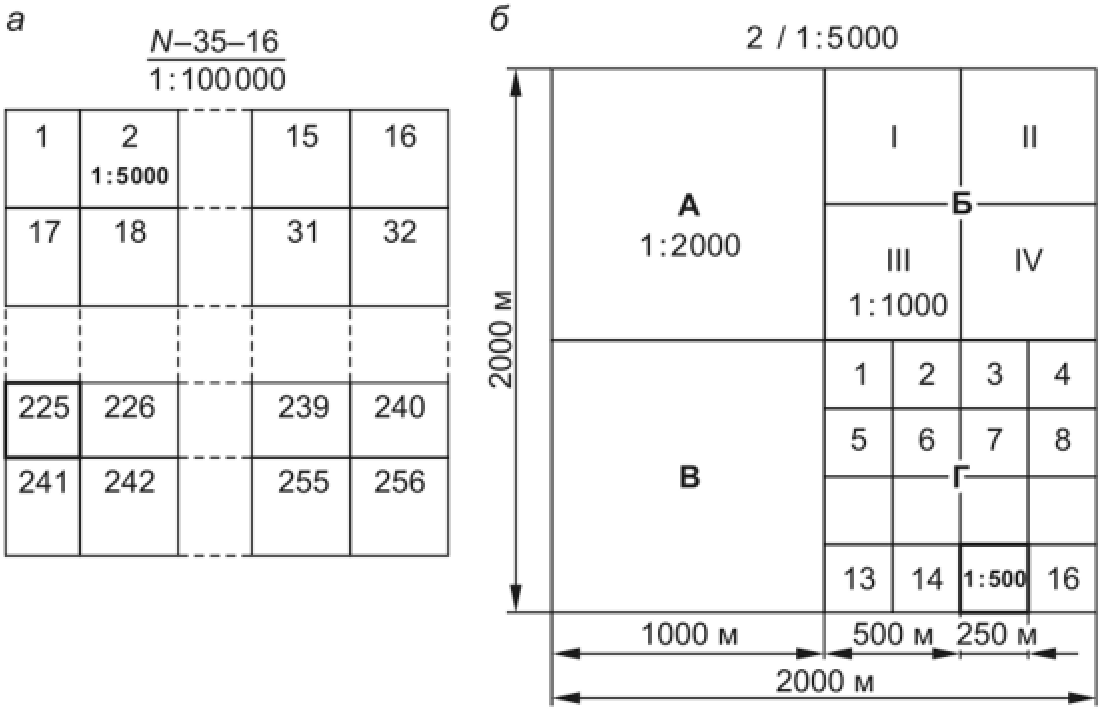


Fig.16 a , b . Layouts of maps at a scale of 1:5000, 1:2000 and 1:1000

At high latitudes, sheets of topographic maps are narrow and inconvenient for practical use. Therefore, it was accepted that to the north of the 60° parallel, sheets of topographic maps are compiled and published in double longitude, and to the north of the 76° parallel - quadruple. Table 1 shows data on the layout of sheets of topographic maps and plans of various scales.

Specific is the layout and nomenclature of marine navigational charts. The dimensions of the sheets and their boundaries are chosen so that each sheet includes a part of the coast or sea that is completely defined in terms of navigation and geographical terms. Therefore sheets of nautical charts are different in size. In addition, sheets of sea charts have mutual overlaps (finds). This is done to make it easier (when laying the path of the ship) to transfer the place of the ship when moving from one sheet to another. Sheets of nautical charts are assigned in each country their serial numbers (called hydrographic and admiralty numbers) as new sheets are produced, regardless of their scale and sea.

The frames of all navigational charts available for the basin and the numbers assigned to the charts (not repeated on the charts issued by the country's hydrographic office) are indicated on a small-scale chart of the basin, which is called *a compilation sheet.*

According to the prefabricated sheet, the provision of the pool with navigation charts is visible, and on it you can select the maps necessary for navigation. The hydrographic services of different countries publish catalogs of maps, in which combined sheets are placed. Chart catalogs are accompanied by sailing directions - navigation manuals that contain descriptions of water basins that explain and supplement sea navigation charts.

Generalization in translation means generalization. *Cartographic generalization* is selection and generalization of objects depicted on the map, according to the purpose and scale maps and features of the mapped area.

The purpose of a card affects its content. For example, a general geographical map and a map for teaching geography at school differ in that the first one contains a lot of geographical objects depicted quite accurately and in detail, while the second one has a much smaller number of objects and they are shown more schematically and larger. The use of large conventional symbols and fonts, and in connection with this a large degree of generalization, are caused by the purpose of the map, which is shown in the classroom at a considerable distance.

The scale of the map largely determines the size of the area depicted on the map. Small scale maps cover large areas. Large-scale maps depict small areas of the earth's surface within separate sheets; large areas require a large number of sheets of such maps. Different spatial coverage leads to a different approach in depicting details. What is an important detail for small plots may be secondary or completely irrelevant for large areas. For example, the administrative map of the district shows in detail the network of local roads, including unpaved ones. On a map of the republic, these roads would be ballast, since it is important to clearly show the road network connecting district and regional centers. It follows that generalization associated with scale, with a decrease in the overall size of the image, is necessary not only because of the lack of space, but also for the transition to generalizing phenomena of a higher level.

To explain the generalization factor - displaying the features of the mapped territory on the map - we point out that the same objects are assessed differently for different landscapes or for depicting different phenomena. For example, wells in the settlements of the European territory of the USSR, as a rule, are not shown on small-scale topographic maps, but on the same maps of desert and semi-desert regions they are the most important element of content. When depicting relief in mountainous areas on maps, a cross-sectional height of two to four times greater than in flat plains is used.

The task of generalization is not only to eliminate redundant information, but also to identify the main, essential characteristics of the mapped phenomena.

Types (principles) of generalization

In the process of generalization, mapped phenomena are selected, their quantitative and qualitative characteristics are summarized, the contours of the depicted objects are graphically generalized and the transition from simple objects to more complex ones is used, their collective designations are used.

The selection of mapped phenomena is due to the fact that the map always displays only some of the phenomena of reality that are significant from the point of view of the purpose of the map, its subject matter, scale and features of the territory. In the course of selection, they are guided by qualifications that establish the levels of selection of objects of different categories. Exclusion qualifications are applied, according to which objects are removed from the map (for example, all rivers less than 1 cm long on the map scale) and electoral qualifications indicating objects that must be depicted on the map (for example, all regional administrative centers).

In the course of selection, a selection rate or “representation rate” is established, i.e. assign the number of objects to be saved, for example, set for a given territory the number of settlements shown on 1 dm ^{2} maps (the number of settlements shown on 1 dm ^{2} maps at a scale of 1: 100,000 varies from 140 to 60 depending on the density and size of settlements points on the ground).

The generalization of the quantitative characteristics of the depicted objects consists in the transition from a continuous scale to a step scale, in the enlargement of the intervals (steps) within which changes in the quantitative characteristics are not reflected on the map. For example, on maps of scales 1:10,000 - 1:100,000, the four-stage gradation of rural settlements (more than 1,000 inhabitants, from 500 to 1,000 inhabitants, from 100 to 500 inhabitants, and less than 100 inhabitants) is replaced by a two-stage gradation when switching to a 1:1 scale map 000,000 (more than 1000 inhabitants and less than 1000 inhabitants).

Generalization of a qualitative characteristic is used to reduce qualitative differences in a given category of objects. For example, this is achieved by replacing fractional classifications with generalized ones (for example, when moving from topographic to geographical maps, they replace special signs for coniferous, deciduous and mixed forests with a single forest sign), as well as by excluding low levels of classification (for example, when characterizing settlements on administrative grounds exclude special signs for district centers and village councils).

Geometric spatial generalization consists in a thoughtful simplification of the planned outlines of the depicted objects - linear, areal - while maintaining the features of the outlines characteristic of this object.

One of the methods of geometric spatial generalization is the use of off-scale conventional signs to depict “point” objects in nature, or the areas of which are not expressed on the scale of the map (geodesic points, road signs, springs).

Replacing individual objects with collective designations, i.e. replacement of individual objects with generalizing designations, for example, replacement of alternating small contours of a bush and a meadow with a generalizing sign of a bush in a meadow without indicating the boundaries of individual contours.

Generalization in the transfer of relief on the map is necessary to identify and display using contour lines and other methods of typical features and characteristics of the surface of the mapped area. The surface of the earth usually has many small irregularities of a particular nature, the abundance of which obscures the general patterns of the structure of larger forms. In the process of generalization, by excluding some elements and landforms, the general features of the relief structure are preserved and depicted on the map.

On maps of different scales of the same territory, generalization in the image of the relief is manifested in an increase in the height of the relief section and as a result of generalization of the contours. Due to this, the map does not depict small landforms that are inside the increased height steps. In table. 2 shows the heights of the relief section, assigned depending on the scale of the topographic map and the characteristics of the territory.

It is very important to preserve the features of the mapped objects, therefore, on the maps, they tend to highlight the structural lines of the relief: watersheds, thalwegs, ridges, and slope toes. This achieves the correct transmission of the relationships of landforms.

Mountain slopes with angular characteristics of dissecting forms are conveyed on maps by angular curves of contour lines in full accordance with nature.

Chapter 5
Practical use of the map (plan)

§ 17. Determining the coordinates of points on the map. Determination of geographical coordinates. Definition of rectangular coordinates.

The position at point *A* in the system of geographical coordinates (φ , λ) will be determined if the latitude of the parallel and the longitude of the meridian passing through this point are known.

The image of the area on a separate sheet of a topographic map is limited by the lines of the inner frame of the map: parallels - from the south and from the north, meridians - from the west and east. At the vertices of the corners of the frame, their latitude and longitude are signed. Inside the map sheet, the lines of meridians and parallels are not plotted. However, they can be built using a special layout of lines in latitude and longitude, which is available outside the inner frame of the topographic map sheet (Fig. 17 *)* .

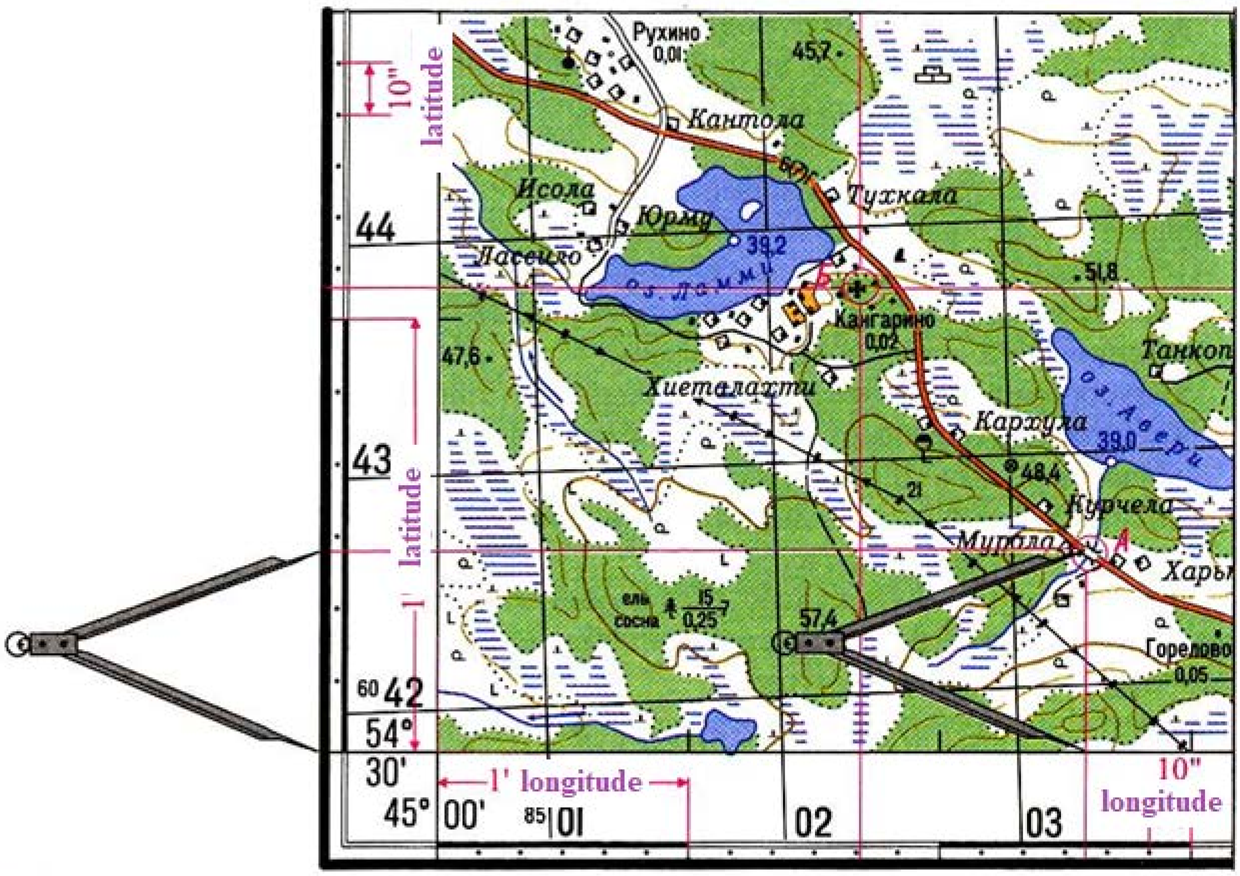


Fig 17. Determining the longitude and latitude of a point on the map

To determine the nirota (φ_A), points *A* are drawn on the map through the minutes of the same name on the western and eastern sides of the frame, the parallel closest to the point from the south. To determine the seconds of latitude from a point *And* they lower the perpendicular *AA* $_{1\ \textit{to this parallel}}$ (Fig. 17) and measure its length *l* in mm. Measure along the minute frame a segment of the parallel corresponding to one minute of latitude - *L* mm. Using the measured values, make up the proportion:

L , mm - 60″ , l , mm - $\Delta\varphi''$

and determine the number of seconds of latitude $\Delta\varphi''$:

$$\Delta\varphi = \frac{60''l}{L} = 0'14'', \qquad (23)$$

which must be added to the latitude of the parallel to obtain the final value of the latitude of point A -

$\varphi_A = 54°41' + \Delta\varphi'' = 54°41'14''$.

Determination of longitude (λ_A) of a point *And* they perform in the same sequence, drawing the meridian closest to the point from the west through the same minutes of the longitude of the northern and southern frames, and lower the perpendicular *AA* $_{2\ \text{onto it}}$, the length of which l_1 (mm) is compared with the length *1 ′* of longitude L_1 (mm), getting value

$$\Delta\lambda'' = \frac{60''l_1}{L_1}, \qquad (24)$$

and finally

$\lambda_A = 18^\circ 01' + \Delta\lambda'' = 18^\circ 01' 12''$.

In order for the accuracy of determining geographic coordinates from topographic maps to be comparable with the accuracy of these maps, calculations using formulas (6) and (7) for maps of scales 1:10,000 - 1:50,000 are made rounded to 0.1″, and for scale maps

1:100,000 - up to 1″.

Definition of rectangular coordinates. To determine the coordinates of a point in the Gauss-Kruger planar rectangular coordinate system, a grid of squares plotted on the map is used. The vertical lines of the grid are parallel to the abscissa axis - the axial meridian of the zone, the horizontal lines are parallel to the ordinate axis - the image of the equator on the plane. The distances between the nearest lines of the coordinate grid (the side of the square) are multiples of a certain number of kilometers on the ground, so the grid of rectangular coordinates is usually called the kilometer grid. On a scale of 1:10,000, 1:25,000, 1:50,000, the side of the grid square is 1 km, on a scale of 1:100,000 - 2 km, on a scale of 1:200,000 - 10 km. On maps of scales 1:500,000 - 1:1,000,000 kilometer grids are not plotted. At the ends (exits) of the grid lines outside the frame of the map sheet, the values of their coordinates in km are signed (Fig. 17). The values of the abscissas and the converted ordinates of the extreme lines on the sheet of the kilometer grid are signed in full (four-digit numbers), and the intermediate ones - with the last two digits - tens and one of kilometers. The zone number in the zonal coordinate system is assigned on the left to the value of the transformed ordinates.

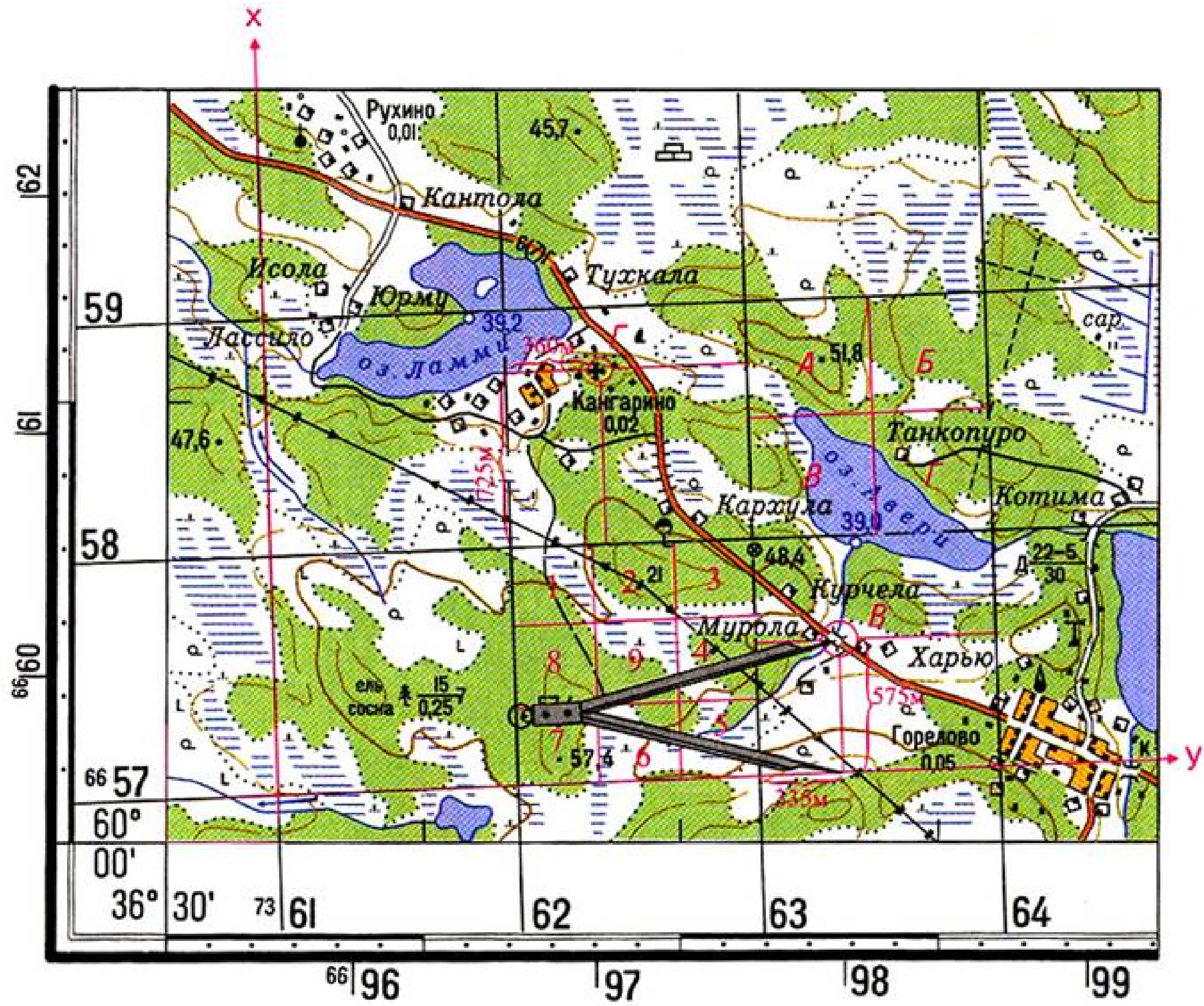


Fig.18. About the definition of rectangular coordinates (*x* , *y*) of a point

To determine the rectangular coordinates (*x* , *y*) of a point *In* are found by digitizing the lines of the kilometer grid, the coordinate values \u200b\u200b(in km) of the southwestern corner of the square in which the point is located. Perpendiculars are lowered from point B to the southern and western sides of the square and their lengths are measured with an accuracy of 0.1 mm, which, taking into account the scale of the map, are expressed in meters. The final value of the coordinates is the sum of the numbers of the value of the grid line signed on the map (in km) and the length of the measured perpendicular (in m). In this case, the abscissa of the point will show the distance of this point from the equator (in m), the ordinate - the number of the zone and the distance from the axial meridian of the zone increased by 500 km (in m), since the ordinate of the axial meridian is +500 km.

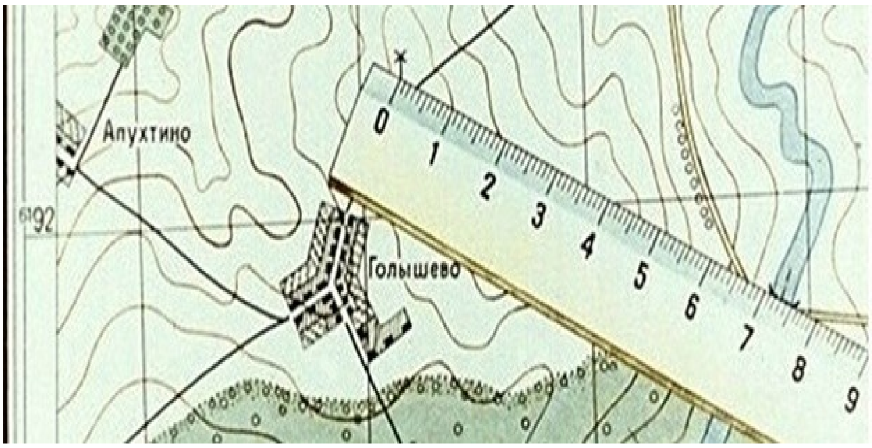
Quite often, the study area is located at the junction of two zones (for example, the Romashkinskoye field), and therefore it becomes necessary to determine rectangular coordinates in an adjacent *zonal coordinate system.* In this case, a square of a kilometer grid is built with a defined point *C* inside it, connecting the outputs of the coordinate lines of the adjacent zone, located on opposite sides of the outer frame of the map. With respect to this square, using the technique described above, determine the rectangular coordinates of the point *C* in the adjacent zone system.

Sometimes, for approximate calculations, it is useful to know the relationship between rectangular and geographic coordinates. An idea of it can be obtained using the following dependencies: it is known that 1 m, as a unit of length, corresponds to 1/40,000,000 of the Paris meridian, or in an angular measure of 0.0324 ″, therefore, it can be assumed with some approximation that the length of the meridian arc 1° corresponds to 111 km. Therefore, to obtain the abscissa of a point from the known latitude of this point, it is sufficient to multiply the value of the latitude of the point, expressed in degrees, by 111 km and vice versa, dividing the known abscissa of the point by 111 km, we obtain the desired value of the latitude of the point in degrees.

As noted above, the map does not depict distances directly measured on the physical surface of the Earth, but their projections - horizontal distances. Measurement on a map of horizontal distances is carried out with a compass - a meter, in the solution of which the distance between the measured points on the map is taken, converted into meters of the terrain using a scale (numerical or linear, or transverse). This performs the following transformation:

$$d' = M d, \quad (25)$$

where d' is the horizontal location of the line on the ground; d is the length of the line on the map, measured on the millimeter scale of the ruler; M is the denominator of the numerical scale of the map.



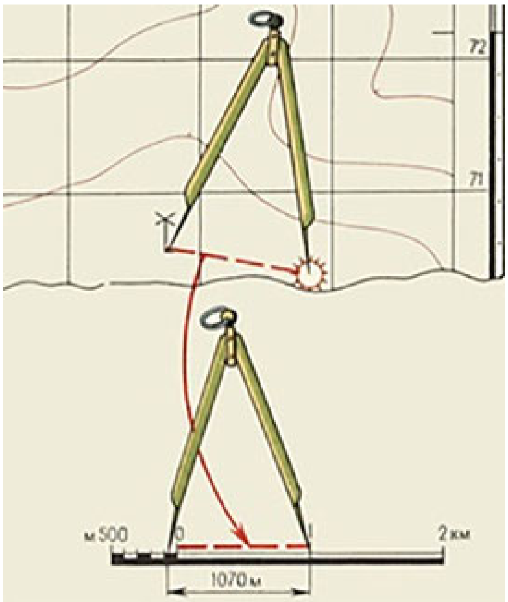


Fig. 19. Determining the distance on the map

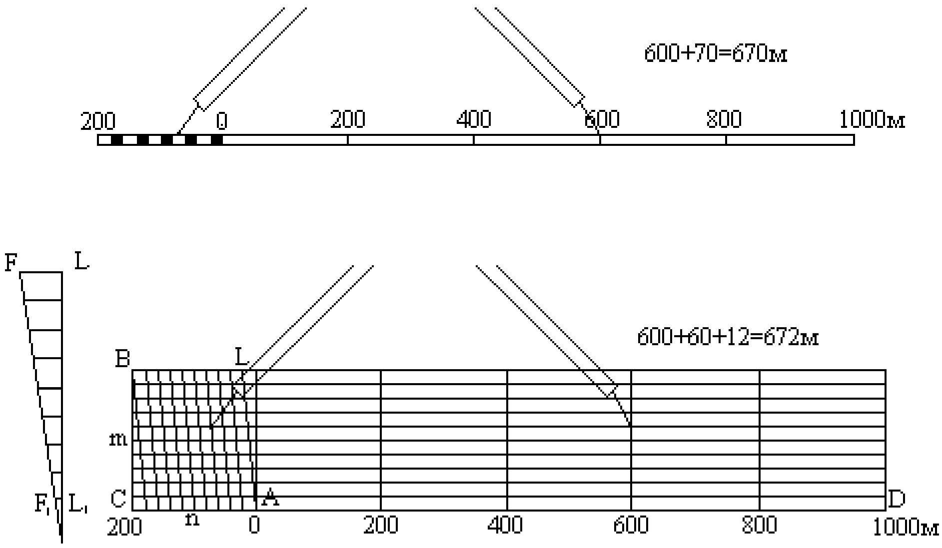


Fig. 20. a) determining the linear distance , b) using a transverse scale.

For example, on a map with a scale of 1:10,000, the length of the segment is *d* = 123.1 mm. On the ground, it will correspond to the laying *d ′* \u003d 123.1 mm 10,000 \u003d 1231 m .

With a large amount of work, in order to avoid calculations associated with the use of a numerical scale (Fig. 20, b), measurements are taken using a linear or, if high accuracy is required, using a transverse scale.

The linear scale is shown in fig. 20 a. Each segment of a linear scale equal to 2 cm is called *the base of the scale* . The labels of the segments are made in accordance with the numerical scale of the map 1:25,000. The first segment is additionally divided into several parts to improve the accuracy of measurements. On fig. 20, *b* , the price of the smallest division is 25 m. The length of the horizontal distance *AB* of the terrain line marked in this figure is

AB \u003d 1500 + 380 \u003d 1880 m.

The transverse scale is shown in fig. 20, *in* . the base of the scale, equal to 2 cm, corresponds to a terrain of 500 m. The first base is divided into 10 parts, each part corresponds to a terrain of 50 m.

Horizontal parallel lines divide perpendicular *OF* into 10 parts. Segment $P_1 F1$ _ corresponds to the terrain 5 m; since $\Delta OP_1 F1$ _ is similar to ΔOPF , then

$$OF_1 = \frac{OF}{10};\ P_1F_1 = \frac{PF}{10} = \frac{50}{10} = 5\,\text{m}\ .$$

To measure the horizontal lines of the terrain on the map, take this line from the map into the solution of a compass - meter; put it on a transverse scale so that one needle falls on the perpendicular to the right of zero, and the other on the inclined line (transversal) to the left of zero, moreover, both needles must be placed on the same horizontal line. Horizontal location of terrain lines *AB* and $A_1 B_1$:

$$AB = 1500\,\text{m} + 50\text{m} \cdot 7 + 5\text{m} \cdot 9 = 1895\text{m};$$

$A_1 B_1$ \u003d 500 m + 50 m · 6 + 5 m · 5.5 \u003d 827.5 m.

If necessary, then according to the horizontal position of the line d' , one can determine the slope distance *S* on the ground by the formula

$$S = \frac{d'}{\cos \nu}, \qquad (26)$$

where ν is the angle of inclination of the line to the horizon.

The error in determining the distance between points on the map using a compass-meter and a scale bar depends on the accuracy of the map and the graphical accuracy of the measurement.

The error in determining the slope distance from the topographic map (using formula (9)) also depends on the error in determining the slope angle ν from the map, however, the influence of the latter will be noticeable only for mountainous terrain conditions, in other cases, the accuracy of the map remains decisive.

However, it should be emphasized that if the position of the points on the map is erroneous, then the distances between these points will be determined erroneously, regardless of the method of determination.

Measurement of the length of curved contours (rivers, winding sections of roads) can be performed using a curvimeter. However, in this case, the measurement results will be very approximate , since the accuracy of the readings depends on the quality of the map paper and the adhesion of the odometer wheel to the paper. More accurate results can be obtained by using a compass for these purposes - a meter with a small solution (step). Having placed one leg of the compass at the starting point, and the other on the contour, they begin to “walk” along the contour, turning the compass in sequence around one of the needles. The total length of the contour is equal to the number of “steps” multiplied by the length of the “step”, plus the remainder, measured on a linear scale. If the curves are smooth, then they are divided into a number of small segments, allowing one to neglect the difference between the length of the chord and the arc in each of them. In this case, the measurement of a curved contour is reduced to the measurement of a broken line.

In connection with the study of offshore oil and gas fields, it may be necessary to measure distances using sea charts. On nautical charts that are built in the Mercator projection, a linear scale is not given. Its role is played by the eastern or western side of the map frame, which are maridians, divided through 1′ in latitude. For sailors, the distance is usually estimated in miles. A nautical mile is the average length of a meridian arc of 1′ in latitude, equal to 1852 m. Therefore, the frames of a nautical chart are actually divided into nautical miles. By determining the distance between two points on the map in minutes of meridian arc, the actual distance is obtained in nautical miles.

If the points *A* and *B* , between which the distance is measured, are located on different meridians, proceed as follows. The compass solution corresponding to the measured distance is transferred to the frame so that both needles of the compass are at the same distance from the ends of the projections of the measured line. To do this, find the middle (point *K*) of the segment *AB* and project it onto the side of the frame (point K_1). From this point, segments are laid on the frame $K_1 A_1$ \ *u003d KA* and $K_1 B_1$ \ *u003d KV* . The distance *AB* in miles will be equal to the difference between the readings of the latitudes of points A_1 and B_1 . in our case it is 55°17′ = 48°25′ = 6°52′= 412′ or 412 miles.

§ 18. Orientation of lines. Determination of the directional angle of the line, geographic and magnetic azimuths

Orientation is usually understood as actions that allow you to find the direction of the line relative to another direction, taken as the original one. In geodesy, the initial directions for orientation are: the true (geographic) meridian, the magnetic meridian and the axial meridian of the zone. In accordance with this, the angles that determine the direction of the lines are called *true azimuth, magnetic azimuth, directional angle.*

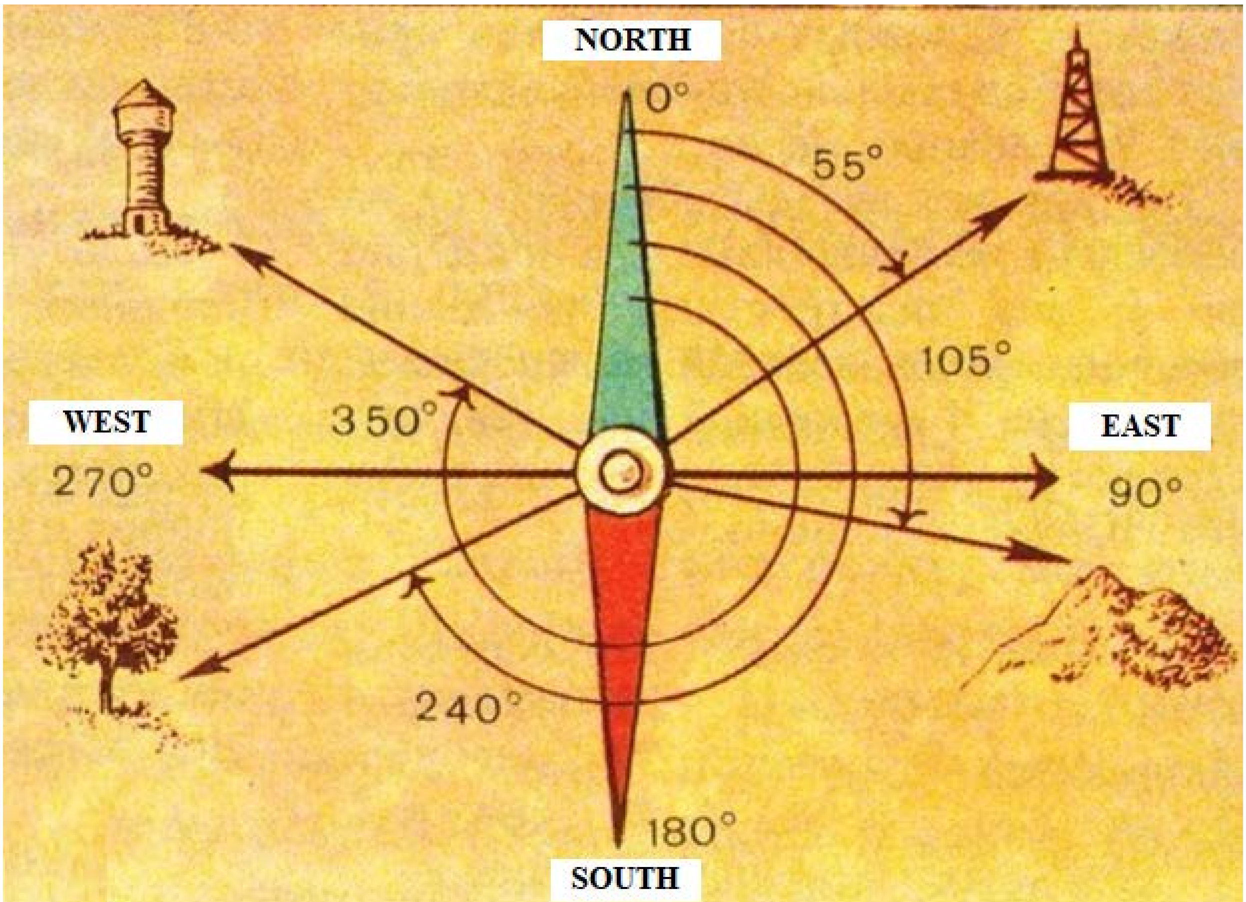


Fig. 21. Line orientation. Determination of the directional angle of the line, geographic and magnetic azimuths

Under *the reference azimuth* (A_{and}) understand the horizontal angle, counted clockwise from the north direction of the true meridian to the direction being determined. Azimuths vary from 0 to 360 ° and can be direct (azimuth line *AB*) and reverse (azimuth line *BA*). Accordingly, the angle A_{AB} - direct azimuth *AB* at the point *A* , angle A_{BA} - reverse azimuth of the same line at point *B* (Fig. 21). The meridian lines are not parallel to each other, so the azimuth of the line that does not coincide with the meridian at each point has a different value. The angle between the direction of the meridians of two given points is called *the angle of approach of the meridians* γ (within the zone - the angle between the direction of the axial meridian and the direction of any other meridian within this zone). The relationship between forward and backward azimuth is given by

$$A_2 \backslash u003d A_1 + 180° + \gamma . \qquad (27)$$

Azimuths are determined from astronomical observations and with the help of gyroscopic instruments.

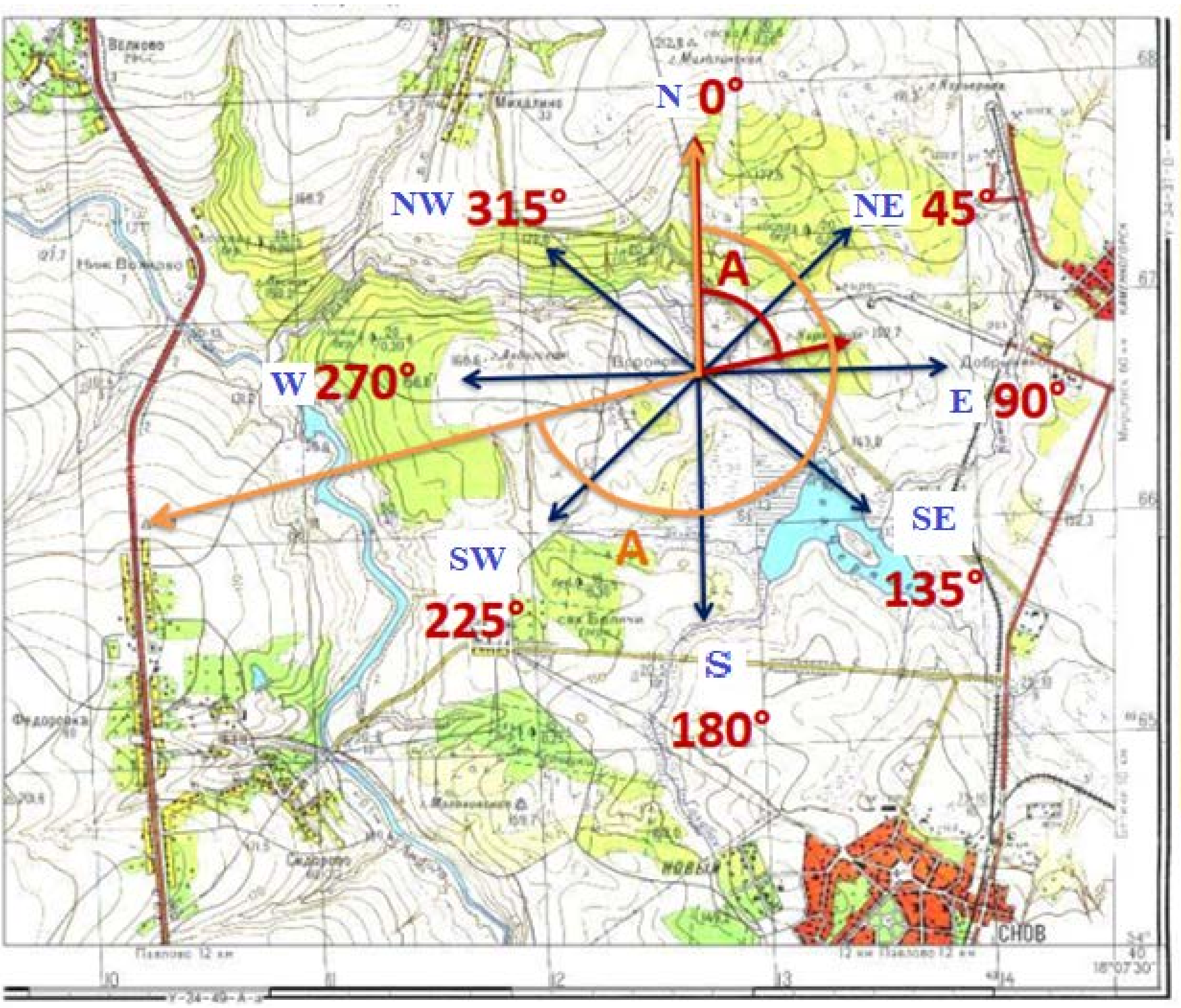


Fig. 22. Line orientation. Determination of the directional angle of the line, geographic and magnetic azimuths on a topographic map

Magnetic azimuth A_m - horizontal angle, counted clockwise from the northern direction of the magnetic meridian to the direction being determined. The direction of the magnetic meridian, fixed by the magnetic needle of the compass, in general, does not coincide with the direction of the true meridian. Therefore, the magnetic azimuth differs from the true one . The deviation of the compass magnetic needle from the direction of the true meridian is called *magnetic declination* . It can be *east* if the northern end of the arrow deviates east from the geographic meridian, and *western* - when the northern end of the arrow deviates from the geographic meridian to the west. Accordingly, the eastern declination is positive (+), the western - negative (-). The relationship between true and magnetic azimuths is expressed by the equation

$$A_u = A_m + \delta, \qquad (28)$$

where δ is the magnetic declination (with its own sign). The magnitude and sign of the declination are usually indicated on the orientation chart (Fig. 22), placed under the south side of the topographic map sheet.

The annual change in declination should also be taken into account.

Information about the magnetic declination of the arrow in a given area can be obtained at the nearest meteorological station or in geophysical observatories, and can also be established using special magnetic field declination maps.

Due to the fact that the magnetic field is changeable, for example, the amplitude of only daily changes is about 15', magnetic azimuths are determined very approximately - to degrees or fractions of them, and it is advisable to use them for approximate orientation on the ground.

When determining the direction of the lines on the map, it is most convenient to use a planar orientation angle, called the directional angle.

Directional angle α is a horizontal angle on the plane, counted clockwise from the northern direction of the axial meridian (or a line parallel to it) to the direction being determined. The relationship between true azimuth and directional angle is established by the equation

$$A_u = \alpha + \gamma , \qquad (2\ 9)$$

where γ is the convergence of the meridians (with its own sign) at the point *And* this is the angle between the images of the axial meridian (the vertical line of the kilometer grid of the map) and the true meridian of this point. Angle γ for points located to the east of the axial meridian, it is positive (+), to the west - negative (-).

The average (for a given map sheet) value and the sign of the convergence of the meridians can be determined using the orientation graph (Fig. 23).

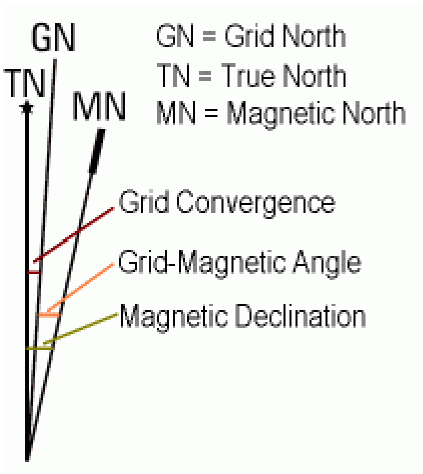


Fig. 23. Orienteering graph

From the coupling equations (28) and (29):

A_u \u003d $A_m + \delta$ and A_u \u003d $\alpha + \gamma$,

on the basis of the equality of their left (and , consequently, right) parts, it is possible to establish dependencies between all terms of the equations, for example, from

$$And_m + \delta = \alpha + \gamma \qquad (3\ 0)$$

should

$$\mathit{And}_{m} \backslash u003d\ \alpha + \gamma - \delta \qquad (31)$$

and

$$\alpha = A_{m} + \delta - \gamma. \qquad (32)$$

An approximate value of γ can be obtained from the formula

$$\gamma_{A} \backslash u003d\ (\lambda_{A} - \lambda_{os}) \sin \varphi_{A}. \qquad (33)$$

Here λ_{A} and φ_{A} - respectively, the geographical latitude and longitude of point *A* ; λ_{os} is the longitude of the axial meridian of the zone in which point *A is located* .

Points are sharp orientation angles, which are sometimes used in practice instead of azimuths and directional angles. Rumb - an acute horizontal angle measured from the nearest (north or south) direction of the meridian to the direction being determined.

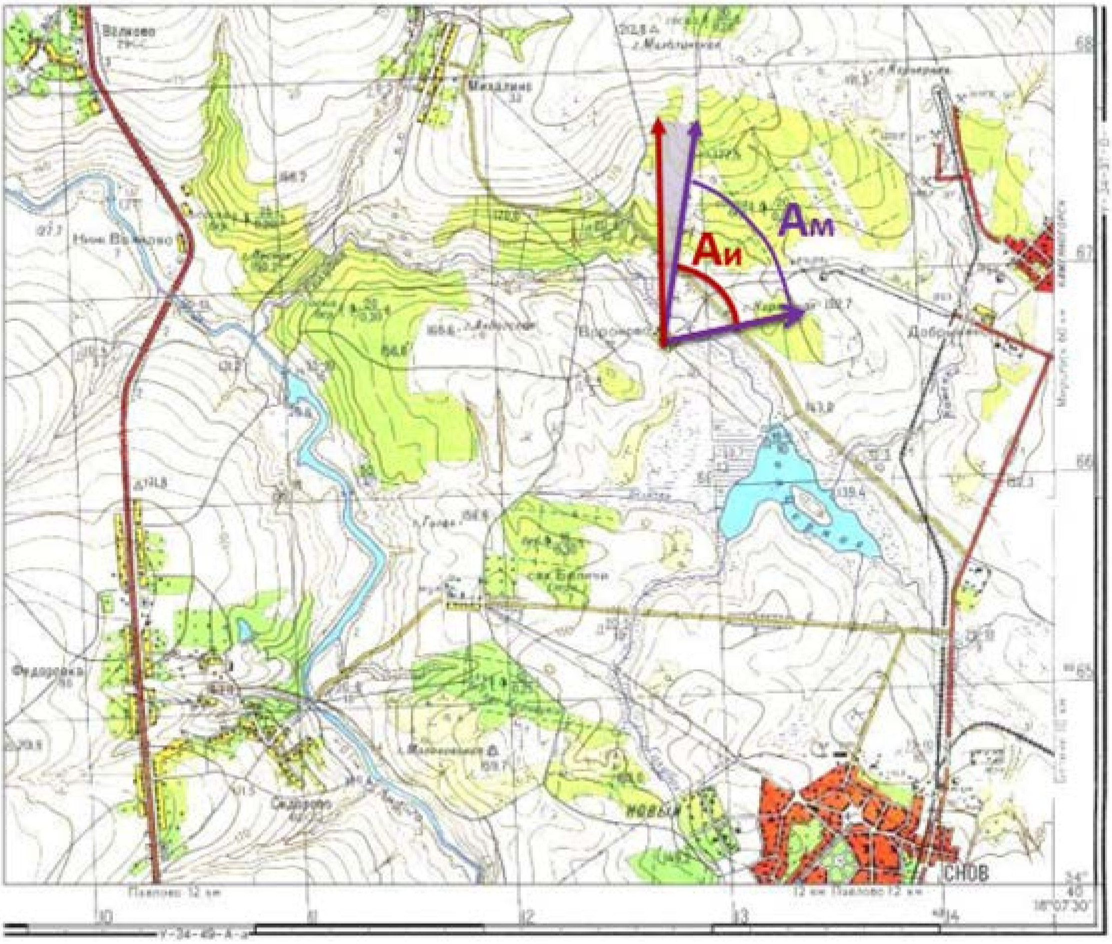


Fig. 24. Determination of the directional angle of the line and magnetic azimuths

Depending on the name of the meridian from which the countdown comes, the directions can be true, magnetic, and simply rhumbs (when counted from the axial meridian).

The relationship between azimuths (direction angles) and points is given in Table 3.

Relationship between azimuths and bearings

Table 3

Azimuth (directional angles), arc. Degree	Rumb	Quarter and rumba name
0 - 90	$r_{\mathrm{I}} = A$	I - CB
90 – 180	$r_{\mathrm{II}} = 180° - A$	II - SE
180 – 270	$r_{\mathrm{III}} = \mathrm{A} - 180°$	III - SW
270 – 360	$r_{\mathrm{IV}} = 360° - A$	IV - NW

The determination of the true azimuths and directional angles of the lines on the topographic map is carried out by measuring, using a protractor, the corresponding reference angles between the directions of the true (or axial) meridian and the line whose direction is being determined. For example, to determine the true azimuth of the line *AB* through the point *And* it is necessary to draw the direction of the true meridian; using the minute divisions of the north and south sides of the frame. If the meridian does not pass directly through the point *A* , draw the meridian closest to it, and then draw a line parallel to the meridian through point *A* and consider it the meridian of point *A.*

By aligning the 0 - 180° line of the protractor with the meridian passing through *A* , from its northern direction clockwise to the line, the direction of which is determined, count the value of the angle, which is the desired azimuth.

To measure the directional angle of a line, it is necessary to draw a line parallel to the vertical line of the coordinate grid through its starting point, and, using a protractor, measure the desired angle clockwise from its northern direction to the direction to be determined. The rhumb of this line will be an acute angle. Since the directional angle at each point of the line is constant, you can use any intersection of the line with the vertical grid line and measure the directional angle at this point.

§ 19. Determination of the altitude position of points. Interpolation.

Determination of the height of the relief section. On modern topographic maps, the terrain is represented by contour lines. The height of the relief section is closely related to the accuracy of the map scale. With the naked eye on the map, a segment of 0.2 mm is perceived, which the eye is able to divide in half. But already half of this segment is perceived as a point. The smallest distance on the map between two successive horizontals (laying) cannot be less than 0.2 mm, therefore, with the denominator of the numerical scale of the map *M* , the angle of inclination (slope steepness) $\boldsymbol{v}$, the height difference between adjacent horizontals (height of the relief section) $\boldsymbol{h}$ must be is equal to

$$h = 0{,}2\ M\ \mathrm{tg}\ v\ . \qquad (34\ )$$

The maximum steepness of the slope, at which the relief on the maps is depicted by contour lines, is equal to $v = 45°$. Hence the normal height of the relief section, m

$$h = 0{,}2\ M\ , \qquad (3\ 5)$$

where *M* is the number of thousands in the denominator of the map scale. For example, for a map with a scale of 1:25,000, it is equal to $h_{25\ \mathrm{thousand}} = 0.2 \cdot 25 = 5$ m. The height of any horizontal line is always a multiple of the height of the relief section.

On topographic maps, the heights of the characteristic points of the area are signed in black: peaks, saddles, brows, etc. From the heights of these points, knowing the height of the relief section and the direction of the slope, it is possible to determine the height of the contour lines. At the same time, it should be remembered that the difference between the height of a point and the height of the horizontal nearest to it is always less than the height of the relief section (or, extremely rarely, equal to it). For example, in fig. 27, *and* with a height of the relief section h = 5 m, a hill is shown, the top of which has a height of 217.5 m, therefore, the height of the contour closest to this point will be 215.

When *determining the heights of terrain points along the contour lines of the* map, there are three options for the location of a point in relation to the contour lines.

1. The point is on the horizontal. The height of this point is obviously equal to the height of the horizontal.

2. The point is located between contour lines (Fig. 25) with different heights. The height of a point is determined by interpolating between the heights of these contours. It can be seen from the figure that

$$H_C = H_r + \Delta h = H_r + \frac{a}{d} h , \qquad (3\ 6)$$

where H_C is the desired height of the point; Δh - exceeding the point C above the horizontal plane; a - a segment from point C to the horizontal closest to it; d - the laying of the slope; h is the height of the relief section; H_r is the horizontal height.

In practice, the value of $\frac{a}{d} h$ is estimated by eye.

3. The point is located inside a closed horizontal or between similar horizontals - on a saddle. The height of a point is determined approximately - it is considered that it is more (or less - depending on the shape of the relief) of the height closest to the horizontal point by 0.5 h .

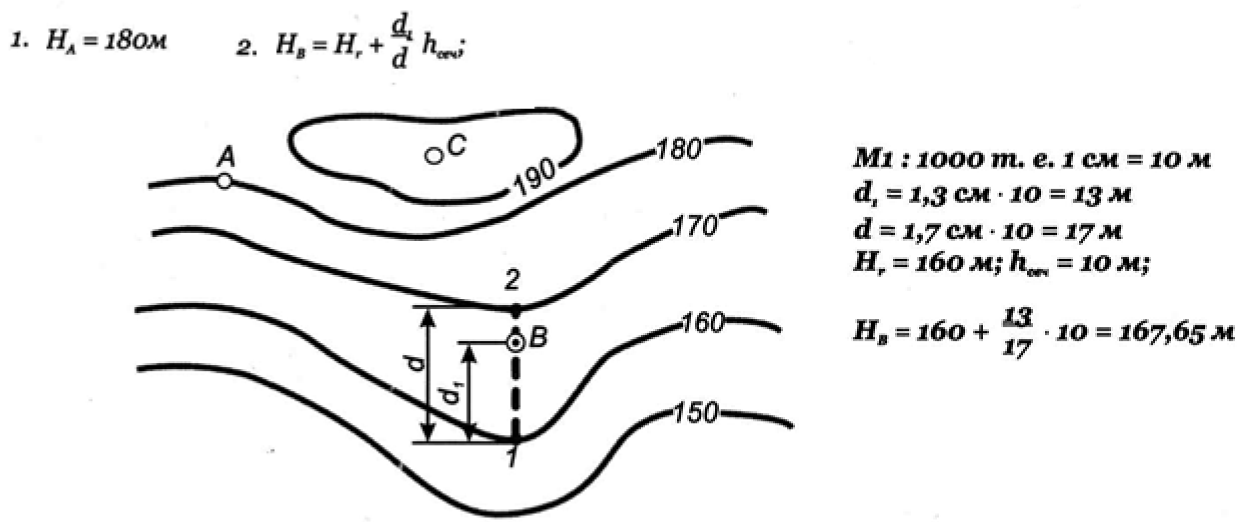


Fig. 25. Determining the heights of terrain points

§ 20. The steepness of the slope .

To determine the *steepness of the slope,* which is characterized by the angle of inclination of the terrain line to the horizontal plane v , measure the magnitude of the laying d and use the expression

$$\operatorname{tg} v = \frac{h}{d}, \qquad (3\ 7)$$

where h is the height of the section, d is the laying. The angle of inclination of the terrain line v is also determined through the tangent:

$$\operatorname{tg} v_1 = \frac{h_1}{ac}, \qquad (3\ 8)$$

where ac is the length of the terrain line, measured on the map between two points, the excess between which is h_1.

The steepness of the slope can be characterized not by the angle of inclination, but by the slope i, which is equal to

$$i = \operatorname{tg} v. \qquad (39)$$

The line slope is usually expressed as a percentage or ppm (thousandths of a unit); it represents the excess per unit length. For example, if i \u003d 0.05 (5%), then this means that for 1 m of length the excess is 5 cm. In ppm, the slope i \u003d 0.015 will be written as 15‰.

To determine the steepness of the slope, a graph is usually used, called the laying graph, which is located under the south side of the map frame. Its construction is based on the formula

$$d = h \operatorname{ctg} v. \qquad (40)$$

To build a graph on a horizontal line, arbitrary but equal segments are sequentially laid aside, signing them with degrees numbers in ascending order of values v. Perpendiculars are left from the obtained points, on which, on the scale of the map, the values \u200b\u200bof d calculated by formula (35) are plotted at the height of the relief section adopted for this map. The ends of the perpendiculars are connected by a smooth curve.

To determine the steepness of the slope (angle of inclination v) in the solution of a compass - meter (Fig. 26), they take the laying between two horizontal lines in the direction of interest and transfer it to the graph so that one leg of the meter is on a horizontal line, the other on a curve. In this case, both legs of the compass lie on a straight line parallel to the lines of the chart. The value of the angle of inclination is determined by the inscription at the horizontal line.

Construction on the map of a line with a given slope. When choosing on the map, for example, a route of movement, the task is set so that the slope of the line of movement does not exceed a certain value.

This line is laid on the map in compliance with two requirements - keeping the slope within the required limits and ensuring the shortest route.

The essence of solving the problem is reduced to determining by calculation the laying corresponding to the given slope, and comparing it with the actual layings - the distances between adjacent horizontals - along the line of the planned route.

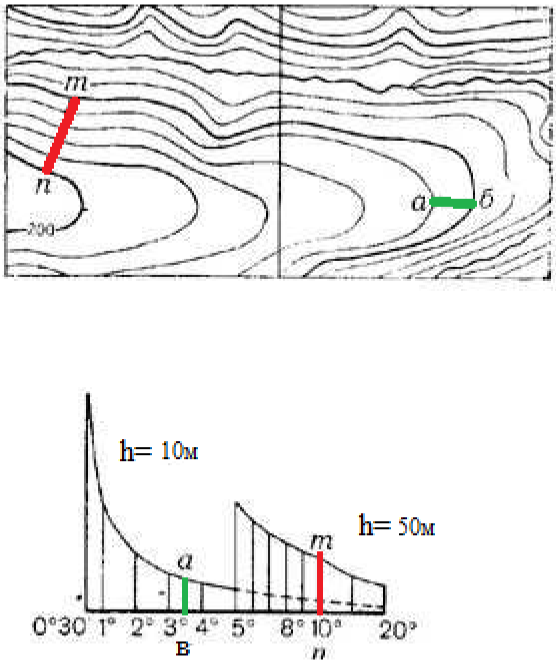


Fig. 26 . Determining the steepness of the slope using the laying chart

For example, on the entire section of the route *AB* , it is necessary to maintain a slope not exceeding 0.050. The laying corresponding to the slope i = 0.050, on the map scale 1: *M* is determined from the expression:

$$d = \frac{h}{i}\frac{1}{M}, \qquad (41)$$

where *h* is the height of the relief section; *M* is the denominator of the numerical scale of the map. For example, for a map with a scale of 1:25,000, the location will be equal to

$$d = \frac{5_{\text{М}}}{0{,}050}\frac{1}{25000} = 4 \text{ мм.}$$

§ 21. Laying schedule.

The resulting value of the laying is taken into the solution of a compass - meter and the distance between the horizontals along the line *AB is checked* (Fig. 27). In those places where the distance between adjacent horizontals is less than the calculated value *d* (sections *A - 1 , 2 - 3*), the slope exceeds the allowable. To reduce the slope, change the direction of movement and ensure that the distance between the horizontals is equal to *d* . Where the distance between the horizontals is greater than *d* , the slope is less than the permissible one and the route coincides with the direction *AB* (sections *1 - 2, 3 - B*).

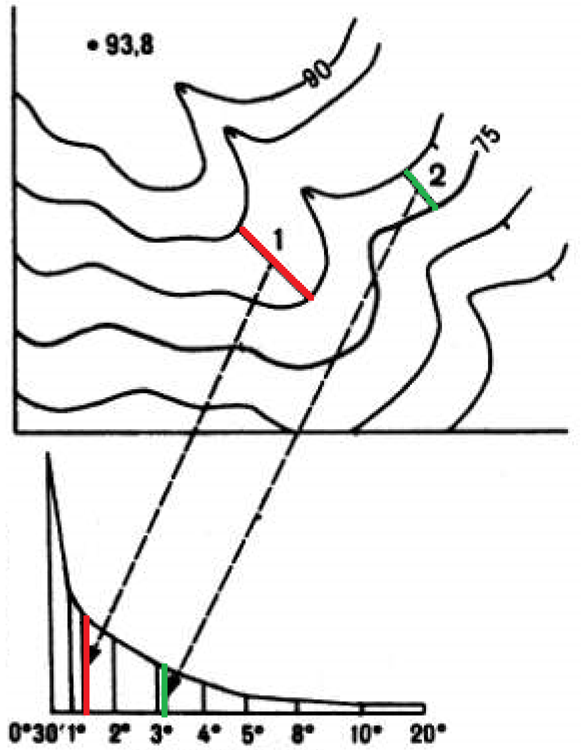


Fig. 27. Laying schedule.

§ 22. Deviations.

The slope (or angle of inclination) is an indicator of the steepness of the slope, which is equal to the ratio of the excess (height difference) to the laying (horizontal distance). Those. the slope is a measure of the steepness of the slope, and the slope is the angle between a given straight horizontal. In general terms, the same thing, but, speaking about the steepness of the slope, it is correct to say either "the slope is equal to ..." or "the angle of inclination is equal to ...".

The slope (or angle of inclination) is calculated either as a percentage, ppm, or decimal fraction through the ratio of the height difference to the horizontal distance (not to be confused with the length of the track) or in degrees through arctan. Percentages (Slope) and Degrees (Slope)! % and ° ! Do not confuse, please!

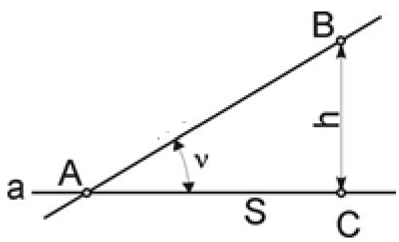


Fig.28. Skate definition

To calculate the slope percentage, you first need to calculate the horizontal distance through the Pythagorean right triangle theorem, knowing the elevation difference and the length of the track (or section of the track). And then divide the height difference by the horizontal distance (to get a decimal fraction) and multiply by 100 (to get a percentage) or 1000 (to get ppm).

Slope i \u003d h / S * 100 (in percent). With a slope of 100%, the horizontal distance of the terrain increases by 1 meter for a vertical rise of 1 m. In this case, the slope angle will be 45 degrees. The slope can be greater than 1000% and indefinitely (with a 90-degree cliff), but not on public ski slopes.

To get the angle of inclination in degrees, it is necessary to find the tangent of the angle of inclination through a similar ratio of excess to the laying h / S and take arctg from it. As a result, i = arctg h/S (in degrees). In degrees, the tilt angle can be modulo 0 to 90°.

§ 23. Building a terrain profile

Building a profile in a given direction. When conducting engineering surveys, it often becomes necessary to characterize the terrain in a particular direction. It is most convenient to obtain this information by constructing a terrain profile on the map between points *A* and *B* - *a vertical section of the earth's surface (Fig. 29).*

The construction begins with the fact that on the profile line *AB* mark the points of its intersection with the characteristic lines of the relief - watersheds, thalwegs, also note the points of change in the steepness of the slopes, determined by the change in the laying of contour lines, etc. Then the line *AB* , called *the base of the profile* , is transferred to the sheet millimeter paper (while maintaining the scale of the map), and from the points marked on it, perpendiculars are restored. On perpendiculars, on a scale 10 times larger than the map scale, distances are plotted corresponding to the heights of the map points plotted on the profile. The points thus obtained are connected by straight lines.

To reduce the length of the perpendiculars, the height of the line of the base of the profile is conditionally taken equal not to zero, but to some round number - a conditional horizon - convenient for constructing a profile (in Fig. 29 - 120m).

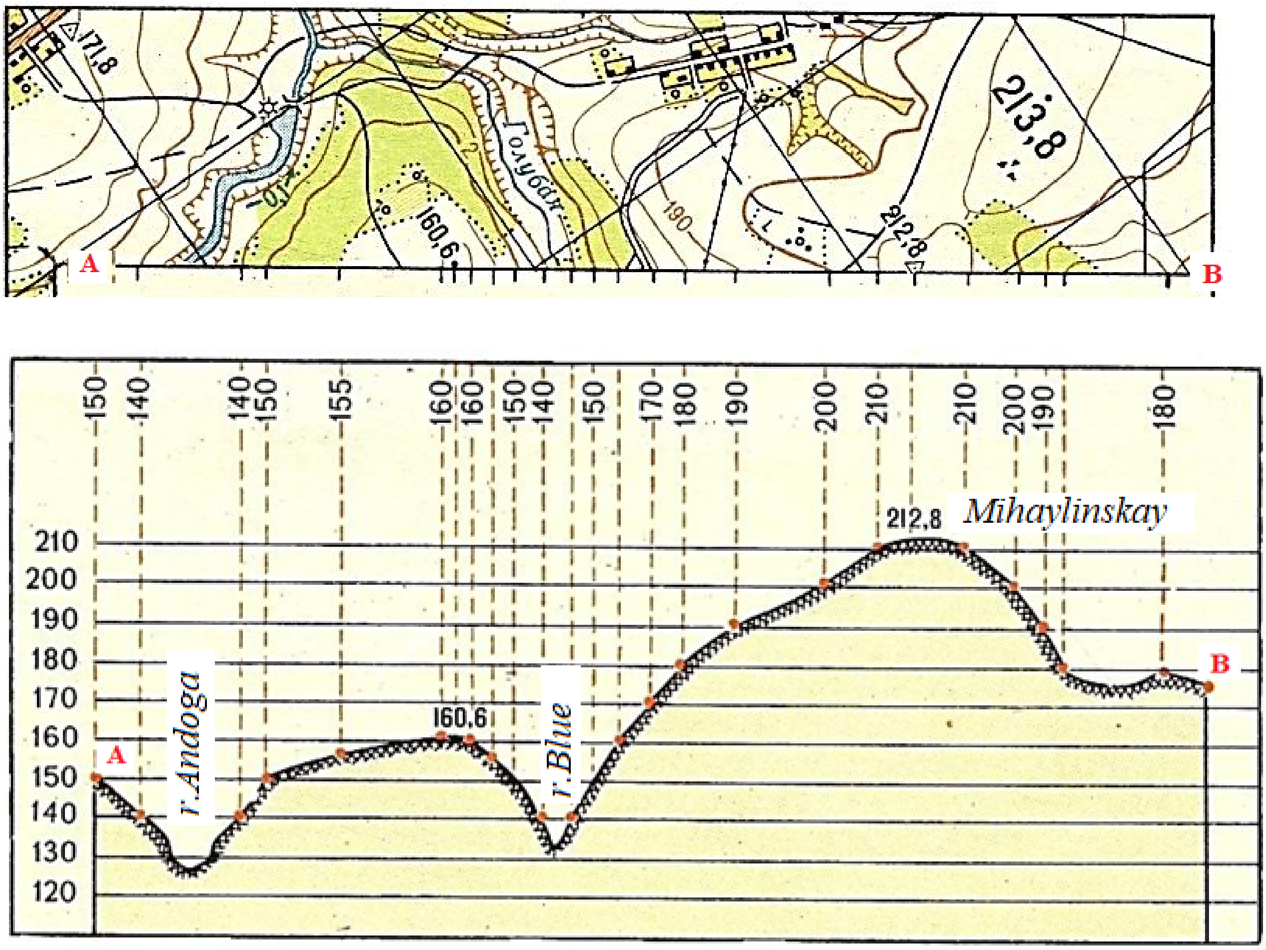


Fig. 29. Building a profile along a given direction AB

§ 24. Determining areas according to the plan.

There are several ways to determine the areas of terrain from a topographic map.

Graphic , in which the area on the map is divided into a number of elementary geometric shapes, and based on the results of measuring the elements of the figures (for example, the height and base of the triangle, the sides of the rectangle, etc.), the area of \u200b\u200beach of them is calculated. The total area of the plot is equal to the sum of the areas of all figures. The graphic method also includes the method of determining areas with palettes.

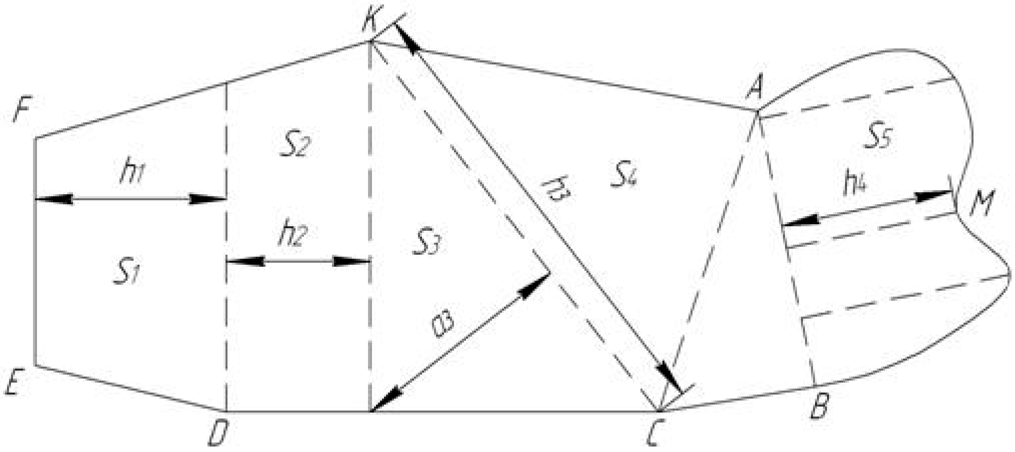


Fig. 30. Graphical method for determining areas. Breakdown into elementary geometric shapes.

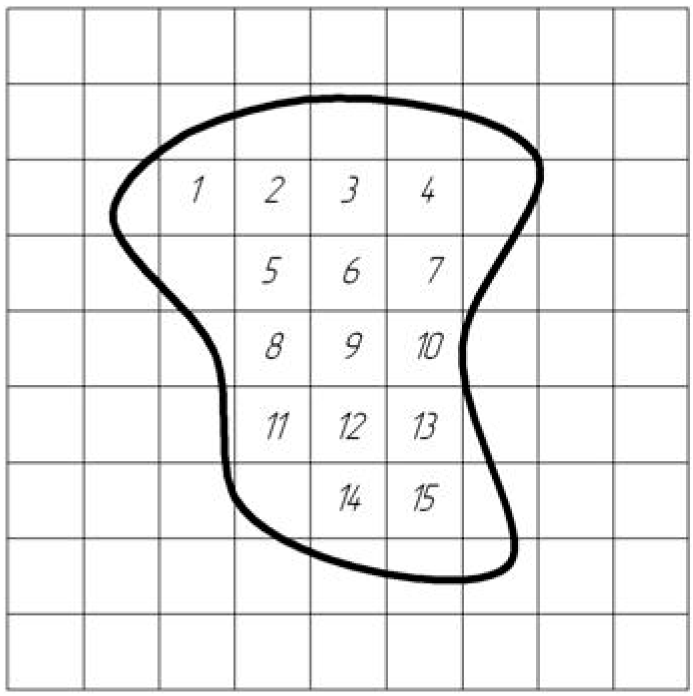


Fig.31. Graphical method for determining areas. Determining the area using the palette method.

Analytical, in which the areas of the plots are calculated from the coordinates of the polygon vertices. These coordinates are usually determined graphically.

Mechanical, in which the area of \u200b\u200bthe site is measured using a special device - a planimeter.

Combinations of several methods are possible. The analytical method has the highest accuracy. The most acceptable in practice is mechanical.

Of interest is one of the varieties of the *graphical method* - the determination of areas using palettes. Let's consider it on the example of using a parallel palette.

A palette with parallel lines has a certain advantage over other types of palettes (dot, square) due to the fact that it does not require counting a large number of dots or squares and their parts. To determine the area of a site, a palette is applied to its contour so that the edge points v , w were in the middle between the parallel lines of the palette. In this case, the sectionstock is divided into figures close to trapezoids with the same heights h , and the palette lines ab , cd ,… are the middle lines of these trapezoids.

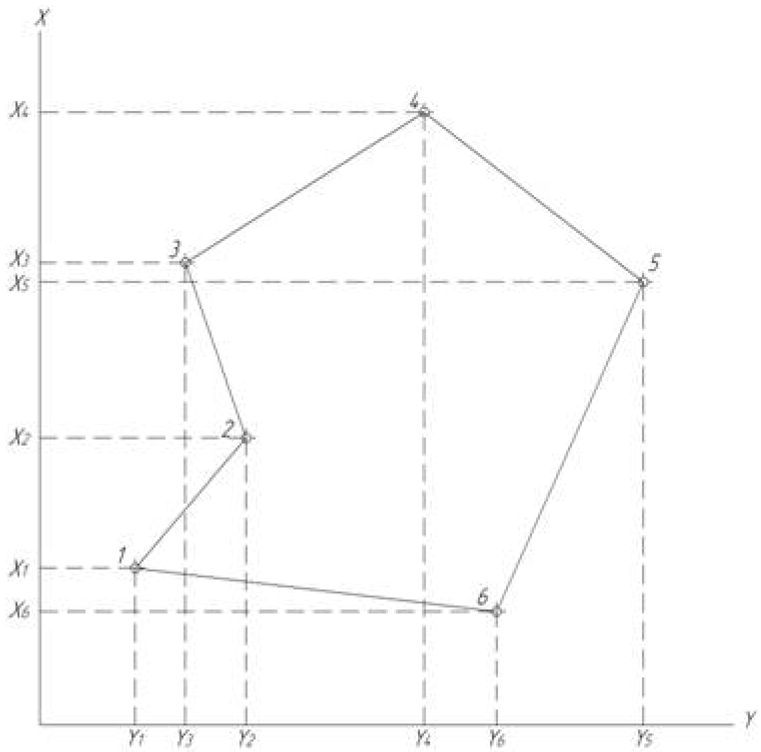


Fig.32. .Calculate the area of a polygon by coordinates

The total area of the plot is equal to the sum of the areas of all trapezoids located inside the contour:

$$P = (abh + cdh +\ldots+ mnh) = h (ab + cd +\ldots+ mn). \qquad (42)$$

The sum of segments *ab* , *cd* . .. is determined by the meter, for which the first segment *ab* is taken into the meter solution, the right meter needle is combined with the beginning of the next segment and, with the left needle stationary, it is taken into the meter solution by moving the right leg, etc. until all segments are collected in the meter solution.

In the *analytical method* of calculating areas, analytical dependences are widely known from geometry. So, for example, if the coordinates *X* , *Y* of the vertices of a closed polygon are known (Fig. 32), then its area can be determined as the algebraic sum of the areas of trapezoids *122'1 '*, *233'2 '*, *133'1 '*. The doubled area of a trapezoid is equal to the product of its height ($y_2 - y_1$) and the sum of the bases ($x_1 + x_2$), therefore

$$2P = (y_2 - y_1)(x_1 + x_2) + (y_3 - y_2)(x_2 + x_3) - (y_3 - y_1)(x_1 + x_3), \qquad (43)$$

after conversion

$$P = (1/2)\,[x_1(y_2 - y_3) + x_2(y_3 - y_1) + x_3(y_1 - y_2)] \qquad (44)$$

or

P \u003d (1/2) $[y_1(x_3 - x_2) + y_2(x_1 - x_3) + y_3(x_2 - x_1)]$. (45)

n vertices :

$$P = (1/2) \sum_{one}^{n} x_i (y_{i+1} - y_{i-1}) \quad (46)$$

or

$$P = (1/2) \sum_{one}^{n} y_i (x_{i-1} - x_{i+1}) \quad (47)$$

where i = 1, 2, 3,..., n .

To control the calculation of the area, they are performed twice - according to both formulas. The accuracy of determining the area in an analytical way depends on the accuracy of determining the coordinates. If the coordinates of the vertices of the theodolite traverse are used, then the relative error in determining the area by this method is 1:1000 - 1:2000.

Determination of areas *mechanically is* carried out using a planimeter. The polar planimeter widely used in practice (Fig. 33) consists of two levers - pole *4* and bypass *7* . At the end of the lever *4* , a load *5* with a needle - a pole is fixed, at the other - a pin *3* with a ball heel, inserted into the socket of the carriage *1 of* the counting mechanism *8 of the* bypass lever *7* . At the other end of the bypass lever there is a bypass spire *6* or a lens with a marked circle and a bypass point, which, in the process of determining the area, are combined with the contour line and slide along it. The counting mechanism consists of a counting wheel *9* , a vernier *10* and a whole revolution counter *2* .

Determining the contour area with a planimeter is carried out in the following sequence:

- install the planimeter on the map in such a way that its pole is outside the measured contour, and the bypass point (spire) is at the initial (any) point of the contour;

- take a count *and* $_1$ by the counting mechanism. The reading always consists of 4 digits: 1st - the smallest digit closest to the index of the revolution counter (whole thousand divisions of the planimeter); 2nd and 3rd - on the counting wheel to the zero vernier stroke (hundreds and tens of divisions); 4th - the number of the matching vernier stroke;

- outline the contour, leading the bypass point along the contour to the starting point, and take the second count *and* $_2$.

The difference between readings *and* $_2$ - *and* $_1$ gives the contour area in planimeter divisions.

Knowing the price of division of the planimeter c (mm ^{2}) or the relative price of division *from* $_1$ (m ^{2}) of the terrain reduced to the scale of the map, determine the area in square millimeters on the map:

$$P = c (u_2 - u_1) \quad (48)$$

or in square meters on the ground

$$P = c_1 (u_2 - u_1). \quad (49)$$

If the pole of the planimeter is set inside the contour, then by analogy with the one described, we obtain the area of the figure:

$$P = c (u_2 - {}_1 u) + q , \quad (50)$$

where q is the planimeter constant, equal to the area of a circle circumscribed by radius r , equal to the distance from the pole to the bypass point, when the plane of the rim of the counting wheel *9* passes through the pole (see Fig. 26). In this position, the movement of the bypass point along the circle of the specified radius does not cause the rotation of the counting wheel *9* and, consequently, the reading of the area, i.e. The area of a circle of this radius is not taken into account by the planimeter.

The division value of the planimeter is determined by tracing a contour with a previously known area:

$$c = \frac{P}{u_2 - u_1}, \qquad (51)$$

where P is in mm ^{2}.

For this purpose, one can use, for example, the squares of the kilometer grid of the map, the area of which is known. They circle the square 3 times, taking at the beginning and after each circle the readings *and* $_1$, *and* $_2$, *and* $_3$, *and* $_4$. Then the differences *and* $_2$ - *and* $_1$ are calculated ; *and* $_3$ - *and* $_2$; *and* $_4$ - *and* $_3$, which should not differ from each other by more than 3 units of the last sign, and calculate the arithmetic mean:

$$u_2 - u_1 = \frac{(u_2 - u_1) + (u_3 - u_2) + (u_4 - u_3)}{3}, \qquad (52)$$

after which the average value of the division price is calculated:

$$c = \frac{P}{u_2 - u_1}, \qquad (53)$$

where P is the known square area of the kilometer grid of the map.

When determining the division price, record the length of the bypass lever R, to which it corresponds. For convenience, you can change the division price *from* and make it “round”, equal, for example, to 10 mm ^{2}. for this, from the proportion R_o \u003d 10 R_1 / c, find the length of the lever corresponding to the desired "round" value, and change the length of the lever, moving the counting mechanism along it to a count equal to R_o.

Examples of entries for determining the division price of the planimeter and determining the area of \u200b\u200bthe contour are given in Table. 4 and 5, respectively.

For example, to obtain the division value c = 10 mm ^{2}, it is necessary to change the length of the bypass lever and make it equal to

$$R_o = \frac{174{,}5 \cdot 10}{9{,}69} = 180{,}1 \text{ мм.}$$

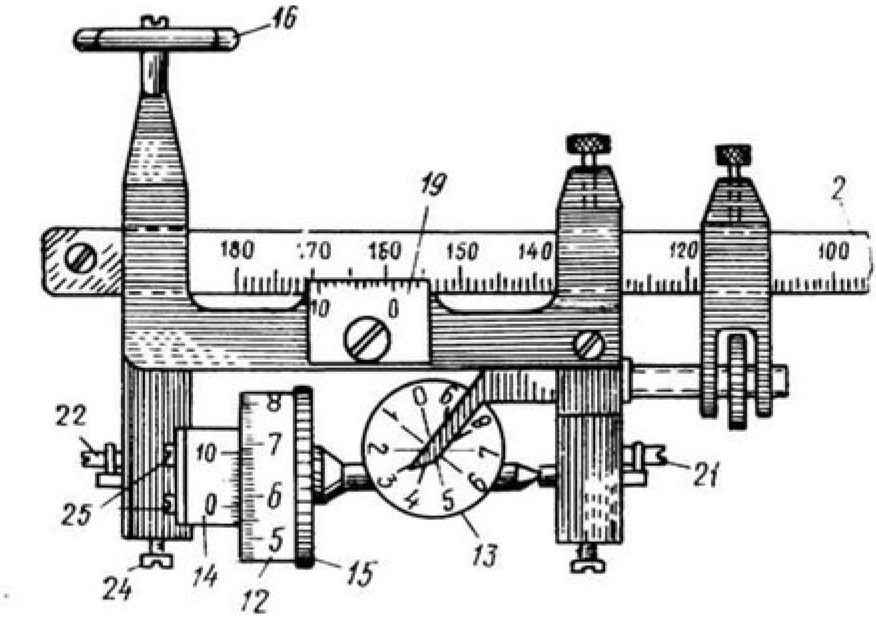


Fig.33. Mechanical planimeter.

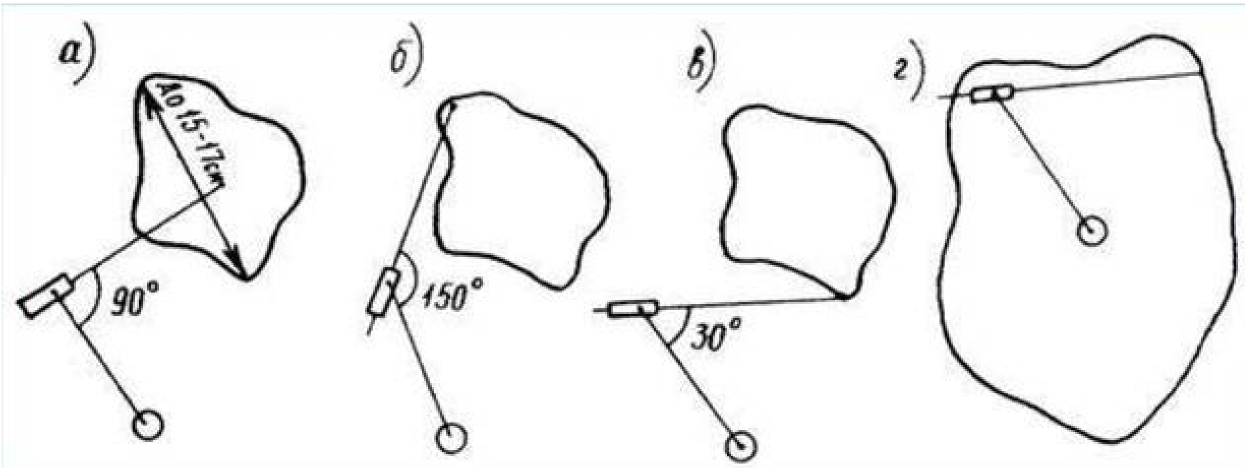


Fig.34. Planimeter setup diagram.

Table 4 **Determining the division value of the planimeter**

Countdown	Count difference	Average	Contour area, mm ^{2}	Planimeter division value	Lever length, mm

4225 1031 square kilometer-

howling map grid c \u003d 10,000 mm ^{2} / 1032 \u003d

5256 1033 1032 M 1:10 000; = 9.69 mm ^{2} 174.5

6289 100 m 100 mm =

1032 10 000 mm2

7321

Fig.35. Electronic planimeter

Table 5 **Determining the contour area**

Countdown *and* $_1$	Count difference	Mean difference	Relative division value for a scale of 1:10,000	Area, ha	Lever length, mm
1204	732		^{1} mm2 - 100 ^{m2}	1000 72.7 /	
1936	733	732.7	10 mm ^{2} - 1000 m ^{2}	/ 10,000 = 73.27	180.1
2669	733		*c* \u003d 1000 m ^{2}		
3402					

In the process of bypassing the contour, the angles between the pole and bypass levers must be at least 30 and not more than 150 °. To determine the planimeter constant q , a contour is drawn at two positions of the pole - “pole outside the contour” and “pole inside the contour” (the dimensions of the contour must provide this possibility). In accordance with expressions (52) and (53), the contour area in the first case is equal to $P = c$ (*and* $_2$ – *and* $_1$), in the second $P = c$ (*and* $'_2$ – *and* $'_1$) + q , whence

$$q = c\,(u_2 - u_1) - c\,(u'_2 - u'_1), \qquad (54)$$

where $u_2 - u_1$ and $u'_2 - u'_1$ are the difference between readings at the pole outside the figure and at the pole inside the figure, respectively.

In critical cases, in order to control and improve the accuracy of the measurement results, contour lines are performed at two positions of the planimeter pole relative to the counting mechanism: the first time - at the position "pole left" (PL), the second time - at the position "pole right" (PP). The positions PL and PP refer to the planimeter pole fixed relative to the bypass lever, if you look along the bypass lever from the side of the readout mechanism.

For reliability, at each position of the pole, the contours are performed twice - once clockwise, the second time - counterclockwise.

Permissible discrepancies between reading differences in planimeter divisions: 2 - with an area of up to 200 units; 3 - with an area from 200 to 2000 units; 4 - with a larger area.

When measuring areas of a small area, it is advisable to reduce the price of division of the planimeter by 2–3 times by changing the length of the bypass lever.

Chapter 6
Angle measurement

§ 25. The principle of measuring angles on the ground.

Geodetic measurements are carried out in order to determine the relative position of points on the earth's surface.

They are linear, angular and high-rise. According to the measurement results, the coordinates of the points of the survey justification are calculated, topographic plans and maps are compiled, plans and profiles of oil and gas facilities are drawn.

Angle measurements are performed to determine the horizontal and vertical angles of lines on the earth's surface and in space.

Altitude measurements (leveling) are performed to determine elevations and heights of points.

Units of measurement.

The unit of linear measurements is the meter $\frac{1}{4000000}$ part of the length of the meridian arc. Introduced in 1799 at the suggestion of the Paris Academy of Sciences. In 1960, at the XI General Conference on Weights and Measures, a new definition of the meter was adopted: the length is 1650763.73 wavelengths of radiation in vacuum, corresponding to the transition between the levels 2 p ^{10} and 2 d ^{5} , the krypton atom - 86.

The units of angular measurements are the circle and its fractions.

Degree $1° = \frac{1\text{окр}}{360}$; $1' = \frac{1^{\circ}}{60}$; $1'' = \frac{1'}{60}$.

Grad (proposed by the Paris Academy of Sciences in 1799)

$$1g^{-} = \frac{1\text{окр}}{400};\ 1' = \frac{1^{g}}{100};\ 1'' = \frac{1'}{100}.$$

Radian - the central angle based on the arc of the weapon, the length of which is equal to its radius

$$\rho = 57.2958° = 3437.75' = 206264.8''.$$

In Russia, goniometers are produced with degree digitization of circles, in some countries devices are produced with digitization of circles in degrees.

§ 26. Theodolite .

Angular measurements are made with a theodolite. Theodolite is an instrument for measuring horizontal and vertical angles. Theodolites vary in accuracy and design.

By accuracy, they distinguish: high-precision T05, T1; precise T2, T5, technical T15, T30. The numbers show the root mean square error of the angle measurement at one time in seconds.

By design, optical, electronic and laser theodolites are distinguished. In optical theodolites, readings on goniometric circles are performed by an observer. In electronic theodolites, a barcode is applied on goniometric circles, the reading is read by a photocell, and the reading value appears digitally on the screen in front of the observer. In addition, when measuring vertical angles (angles of inclination), some designs use a level to set the horizon line, while others use a compensator (pendulum device).

Some types of electronic theodolites have a laser built into the optical system, which is used during stakeout work. Table 6 shows the main characteristics of some modern theodolites.

Table 6

Specifications	Firm and model					
	LDT50 "SOKKIA"	DT510 “SOKKIA”	3T2 KP UOMZ	3TKP UOMZ	4T15P UOMZ	4T30P UOMZ
Type of theodolite	electronic	electronic	optical	optical	optical	optical
Multiplier	30	30	30	30	20	20
Mean square angle measurement error in one go	5	5	2	5	20	15
Compensator/min.	±3	±3	4	4	no	no
range	dual	axle	dual	axle		
Display	4 lines ×20	2 lines ×8	characters	characters	two screens	two screens
Presence of a laser	yes	no	no	no	no	no
Weight, kg	5.7	4.7	4.7	4.7	3.5	3.
Warranty period	2 years	2 years	2 years	2 years	2 years	2 years

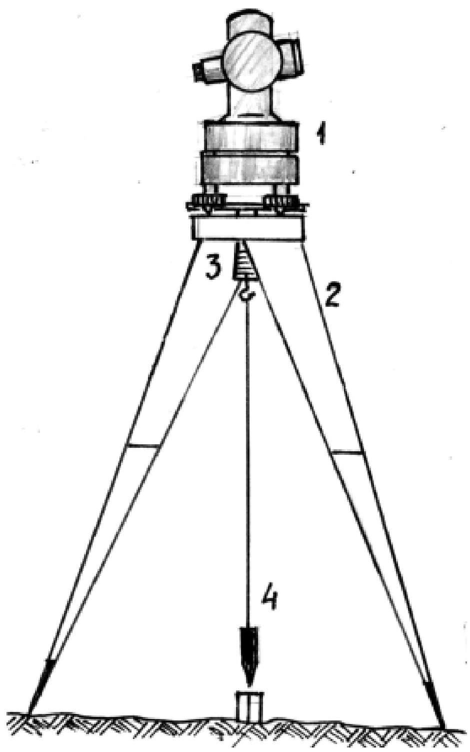


Fig. 36. Installation of theodalite.

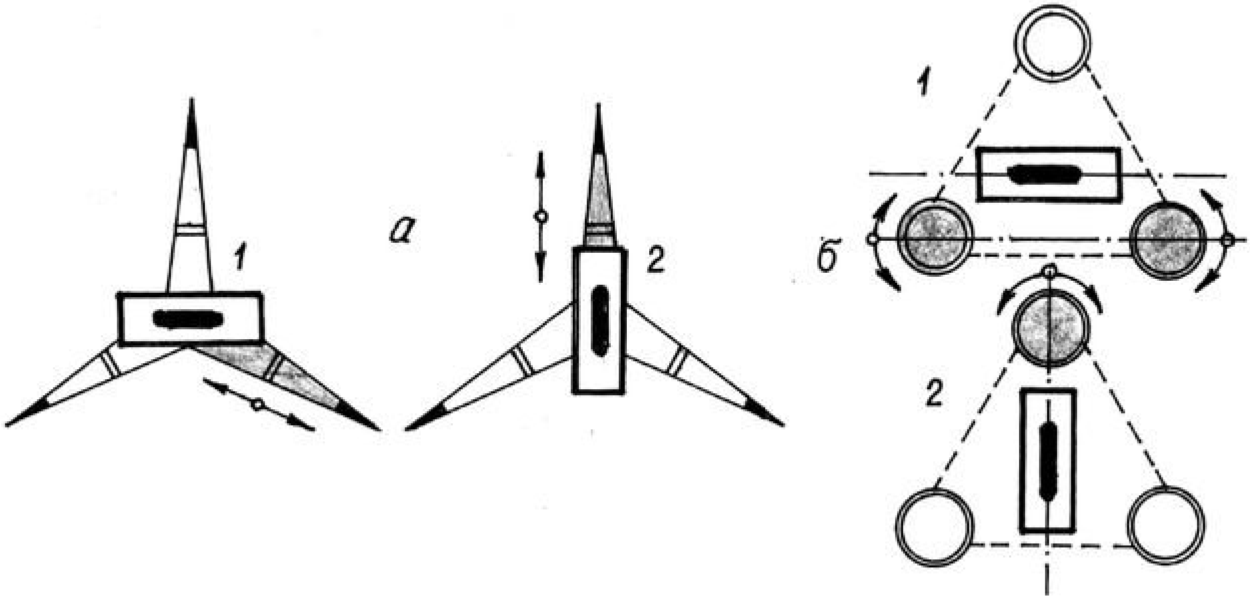


Fig.37. Leveling theodolite a - preliminary leveling with tripod legs; b - leveling with lifting screws of the stand

The vertical axis of the theodolite is set in a vertical position (and the plane of the limb in a horizontal position) according to the cylindrical level *by* turning the lifting screws *3* (total 3 screws) (Fig. 37 b).

The theodolite is mounted on a tripod 2 and fixed on it with a set screw 3.

Theodolite centering, i.e. the installation of the theodolite so that its vertical axis passes through the top of the measured angle is carried out using a plumb line 4 (or for the T30 theodolite using a telescope directed vertically down).

To measure tilt angles (vertical angles), the theodolite has a vertical circle.

In the process of measuring the horizontal angle by rotating the alidade and tube sequentially point the telescope at points *A* and *C* of the terrain and take readings along the limb using a reading device, the difference between these readings is the value of the horizontal angle.

spotting scope

Spotting scopes are used in geodetic instruments to sight at distant objects. Sometimes pipes with a reverse image are used, sometimes they are used with a direct image.

The telescope in fig. 34, **but** has a lens *1* , a focusing lens *2* , which moves with the help of a rack *3* . The rays coming from the observed object are limited by the diaphragm *4* and are focused in the plane of the grid of threads *5* , then they enter the eyepiece *7* . The grid of threads is made in the form of strokes on a glass plate and is fixed in the pipe with adjusting screws *6* . On fig. 34b shows a grid of strokes: *2* and *3* are horizontal and vertical strokes, respectively, *1* is a bisector, and *4* is rangefinding strokes.

An imaginary line connecting the optical center of the lens and the cross hairs of the reticle is called the line of *sight of the* tube.

To prepare the tube for observations, the tube is set “by the eye” - point the tube at a light background and turn the eyepiece ring, achieving a clear image of the grid lines. They install the pipe “on the subject” - point the pipe at the object and, rotating the rack, achieve a clear image of the object. When moving the eye at the eyepiece, the crosshairs of the reticle should not leave the image of the observed target. Otherwise, there is a parallax of the grid of threads, which occurs due to the fact that the image plane does not coincide with the plane of the grid. Parallax is eliminated by small turns of the chimney and the eyepiece ring.

Some characteristics of the telescope. The visible or angular magnification of the telescope is the ratio of the angle β, under which the image of the object under consideration is visible through the telescope, to the angle α, under which the object is visible to the naked eye, i.e.

$$\Gamma = \frac{\beta}{\alpha} \text{ or } \Gamma = \frac{f_{об}}{f_{ок}}, (56)$$

where f about is the focal length of the lens; f ok - the focal length of the eyepiece.

The magnification of the telescope can also be determined by the formula

$$\Gamma = \frac{D}{d}, (57)$$

where D is the diameter of the lens inlet; d is the diameter of the pipe outlet.

It is believed that the sighting error with the naked eye is 1 ′. The sighting error through the telescope m_v decreases in proportion to the increase in the telescope

$$mv = \frac{60''}{\Gamma}. \_ (58)$$

For technical theodolites $\Gamma = 20^{\times}$, $m_v = 3''$.

The field of view of the pipe is the space visible through the pipe when it is stationary. It is limited by the diameter of the pipe diaphragm. The pipe field of view angle ε (in angular degrees) can be determined by the formula

$$\varepsilon = \frac{38{,}2^{\circ}}{\Gamma}. (59)$$

The greater the magnification of the tube, the smaller the angle of view. For a spotting scope, the magnification of which is $\Gamma = 20^{\times}$, the field of view angle is $\varepsilon \approx 2°$.

Theodolite survey results are recorded (registered) in the outline, or in several theodolite survey outlines

An outline in geodesy (and not only) is a schematic drawing (sketch) of the object under study, made “in the field” (that is, directly on the object, in the field) by hand.

If the area to be filmed is small, then one general outline can be drawn up for the entire area. But usually the outlines are compiled for each individual fragment of the site, filmed only from two or three nearby points and sides of the survey justification.

The outline of the theodolite survey depicts the corresponding points and sides of the survey justification, the entire situation taken from them, and, most importantly, the results of all field geodetic measurements are recorded, shown in such a way that it is clearly clear in what way and from which points and sides of the survey justification the survey of one or more other contour point of the area.

The outline of the theodolite survey is the only field document containing its results, and therefore it must be drawn up correctly and accurately, so that any other person (not just the surveyor) can plan the filmed situation from this outline. And of course, the outlines must not be lost: this will devalue all the relevant field work.

Linear measurements

Line length measurements are performed by mechanical, physical-optical and electromagnetic devices.

Mechanical measuring instruments. These include wands, invar wires, ribbons and tape measures. The accuracy of line measurements by the indicated devices is characterized by relative errors, respectively $\frac{1}{1000000}$; $\frac{1}{10000}$-$\frac{1}{50000}$ $\left(\text{до } \frac{1}{1000000}\right)$; $\frac{1}{1000}$-$\frac{1}{5000}$ $\left(\text{до } \frac{1}{10000}\right)$.

In measurements for technical purposes, tapes of 20 and 24 meters in length and tape measures of 10, 20, 30 and 50 meters in length are most often used.

Before starting work, a measuring device is compared, that is, compared with a control measuring device. As a result of the comparison, the correction for comparation is known, which is taken into account in the measurements.

On the ground, the end points of the measured lines are fixed, depending on the duration and importance of the work, with wooden stakes, metal pins or concrete monoliths.

In the opening of the line (a vertical plane passing through the end points of the line), milestones are set (on the plain after about 100 m, and in rough terrain - within line of sight). Hanging is performed by eye, in some cases with the help of a theodolite, setting milestones from the far end of the line "towards you". To calculate horizontal distances, the slope angles of the terrain are measured. The line is measured twice - in the forward and reverse directions. The line length is determined by the formula

$$D = l \cdot n + r, \quad (60)$$

where l is the length of the measuring instrument,

n - the number of positions of the measuring device in the measured line,

r is the remainder.

Horizontal distances are calculated by the well-known formula

$$d = D \cos v \qquad (61)$$

(v is the angle of inclination of the terrain).

The accuracy of measuring lines with a measuring tape, depending on the conditions, is characterized by relative errors (1/1000 - 1/3000).

Optical rangefinders. The basis for determining the distance with optical rangefinders is the solution of the *AMS parallactic triangle* (Fig. 37), in which φ is the parallactic angle, β *is* the base, and D is the distance to be determined. From fig. 37 follows:

$$D = \frac{в}{2} \cdot \frac{1}{tg\frac{\varphi}{2}} \quad (62)$$

Considering that the angle φ is small (usually does not exceed 1°), we can write:

$$D = \frac{в\rho}{\varphi}, \quad (63)$$

where ρ is a radian.

Depending on the solution of the parallactic triangle, the following types of optical rangefinders are distinguished:

- with constant parallax angle and measured base;
- with constant base and measured parallax angle;
- with measured angle and measured base.

Thread rangefinder. The filament rangefinder refers to rangefinders with a constant parallax angle and a measured base. It is available in the telescopes of most geodetic instruments and is a grid strokes: upper *a* and lower b , providing a constant parallactic angle. To measure the distance, a device is installed at one end of the line, and a rangefinder rod at the other.

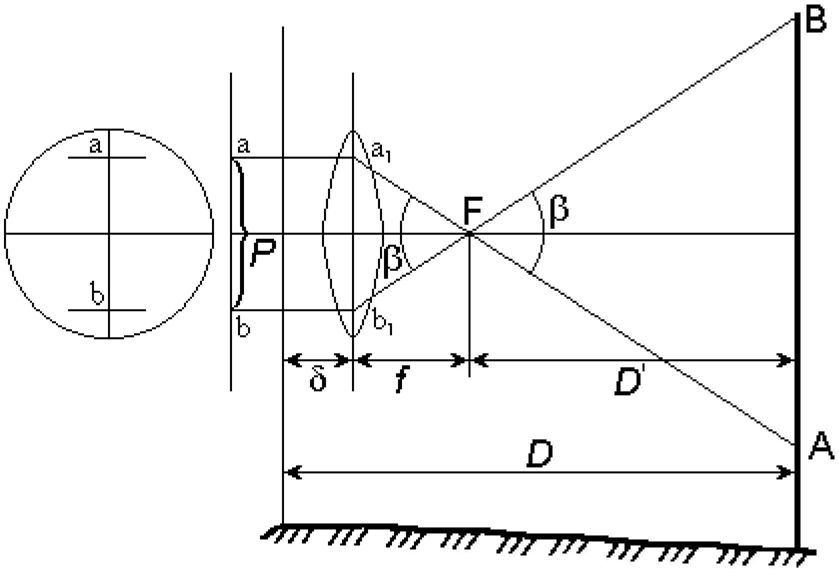


Fig.37 Thread rangefinder..

Rays from strokes *a and* b *pass* through the lens, intersect at the front focus of the lens *F '* and cross the staff at points *A* and *B.* The distance between points *B* and *A* (base) is the difference between the readings on the rangefinder strokes, denoted by *n* . The length of the measured line can be determined from the expression

$$D = D' + f + \delta \text{ , (64)}$$

where *D '* is the distance from point *F '* to the rail,

f is the front focal length of the lens,

δ is the distance from the lens to the vertical axis of rotation of the device.

From the similarity of triangles *ABF ' a ' to F '* we can write

$$\frac{D'}{n} = \frac{f}{p} \qquad \text{or} \qquad D' = \frac{f}{p} \cdot n \text{ , (65)}$$

p = a'b ' _ _

where *p* is the distance between the rangefinding strokes.

Denote by $k = \frac{f}{p}$ is the rangefinder coefficient and through $c = (f + \delta)$ is the rangefinder constant correction, then

$$D = k \cdot n + c \text{ . (66)}$$

In modern geodetic instruments $k = 100$ and $c \approx 0$.

This is convenient for measurements - the reading on the rail in centimeters is easy to translate into distance meters.

Line measurements with a filament rangefinder are performed with a relative error of 1/100 - 1/300.

Reduction to the horizon of distances measured with a filament rangefinder. When measuring distances on the ground with a filament rangefinder, it is necessary to take into account the angle of inclination (practically at inclination angles of more than 2 °).

When sighting on the rail, a reading is obtained $n = AB$. To determine the slope distance D , it is necessary that the staff be at right angles to the line of sight. In this position, the reading on the rail will be equal to $n' = A'B'$. But it should

$n' = n \cdot \cos\nu$ whence $D = n' \cdot k + c$ and $D = n \cdot k \cdot \cos\nu + c$. (67)

The horizontal position is determined by the formula

$d = D \cdot \cos\nu$ or $d = n \cdot k \cdot \cos^2\nu + c \cdot \cos\nu$. (68)

If the value of c is close to zero, then the horizontal distance of the lines measured by a filament rangefinder can be determined by the formula

$$d = n \cdot k \cdot \cos^2\nu . \quad (69)$$

Electromagnetic rangefinders. By electromagnetic rangefinders are meant devices for measuring the distance in time of propagation of electromagnetic oscillations between the end points of the line.

Depending on the range of electromagnetic oscillations used, light range finders and radio range finders are distinguished. Depending on the nature of electromagnetic radiation, pulse and phase rangefinders are distinguished.

Pulse rangefinder works as follows. A rangefinder is installed at one end of the measured line, and a reflector at the other. The rangefinder has a transmitter, receiver and time indicator. The transmitter emits an electromagnetic pulse (radio or light) towards the reflector. The pulse after reflection returns to the receiver. The time indicator measures the time τ of the impulse passing along the distance back and forth. The distance is determined by the formula

$$D = \frac{1}{2} \cdot c \cdot \tau , \quad (70)$$

where c is the propagation velocity of electromagnetic oscillations in the atmosphere, determined by the formula

$$c = \frac{c_0}{n}, \quad (71)$$

where c_o is the propagation velocity of electromagnetic oscillations in vacuum, equal to 299792458 m/s.

n is the refractive index of air.

Pulse rangefinders are used in aerial photography, like altimeters, to determine the flight altitude at the time of photographing. When measuring, the reflector of impulses is the surface of the earth. The error in determining the height by radio altimeters in flat areas is about 1 - 2 m. In mountainous areas, radio altimeters are not used due to large errors in determining the height arising from the reflection of radio waves not from the point above which the aircraft is located, but from the peaks closest to it.

In this rangefinder, the propagation time of electromagnetic oscillations along the distance and back τ is measured indirectly, after comparing the phase of continuous oscillations that go to a distance and return after reflection.

The rangefinder transmitter emits light in the direction of the reflector, modulated in amplitude by the frequency f . After reflection, these oscillations arrive at the receiver.

The phase meter compares the phase of the signal going to the distance, φ_1 , and the phase of the signal coming from the distance, φ_2.

The phase of the outgoing signal is equal to

$$\varphi_1 = \omega t + \varphi_0, \quad (72)$$

where φ_0 and φ_1 are the phases of the harmonic oscillation at the initial moment and time t; ω is the angular frequency of oscillations. It is known that $\omega = 2\pi / T = 2\pi f$. Consequently,

$$\varphi_1 = 2\pi f t + \varphi_0 \quad (73)$$

The phase of the signal coming from a distance is determined by the expression

$$\varphi_2 = 2\pi f(t - \tau) + \varphi_0. \quad (74)$$

The phase difference of these oscillations, determined by the phase meter, is equal to

$\Delta \varphi$ \ *u003d* $\varphi_1 - \varphi_2$ \u003d $2\pi f \tau$, whence $\tau = \dfrac{\Delta\varphi}{2\pi f}$. (75)

With a known value of τ, the distance D is determined by formula (70).

There is one feature of the phase rangefinder. The phase difference generally consists of an integer number of periods N and a fraction of the period $\Delta\varphi$. The value of $\Delta\varphi$ is determined by the phase meter within one period. For a time equal to one period, the oscillation propagates over a distance equal to the wavelength $\lambda = cT$. Therefore, by $\Delta\varphi$ one can uniquely determine the distance not exceeding 1/2 λ. To determine N, measurements are made at several frequencies. First, an approximate distance within $\frac{1}{2}\lambda$ is determined at a low frequency, then it is refined at higher frequencies (the number of periods will be known). Usually enough measurements at three frequencies. The applied modulation frequencies are in the range from 10 to 40 MHz.

The measurement accuracy of phase rangefinders largely depends on the choice of wavelength. For short waves, the influence of the atmosphere can be determined more accurately. But the use of these waves causes difficulties in measuring the phase difference and resolving ambiguity. In phase light rangefinders, this contradiction is resolved due to the amplitude modulation of light waves, in which waves of low (measuring) frequency are superimposed on waves of high (carrier) frequency. The carrier frequency makes it possible to sufficiently reliably take into account the influence of the atmosphere, and the measuring frequency facilitates the determination of phase cycle domers and the resolution of ambiguity.

In the practice of engineering and geodetic works, laser non-reflective rangefinders have become widespread, which measure the distance to any (practically) object without installing a reflector on it.

The main feature of laser rangefinders in comparison with light rangefinders is a large range of action and the ability in some cases to determine the distance to objects without installing reflectors on them. In this case, the reflector is the surface of the object itself. With the use of a reflector, the range of the devices increases.

Key features of some laser reflectorless rangefinders

Table 7

Device brand	Manufacturer	Range without reflection/	Accuracy	Weight, kg

		with reflect		
Data Disto	Leica Switzerland	30 m / 100 m	3 mm	0.6
power disto	Leica Switzerland	60 m / 140 m	3 mm	0.67
Minimeter MM30	Sokkia Japan	30 m / 100 m	3 mm	0.58
LEM 30	Jenoptik Germany	30 m / 100 m	3 mm	0.36
PULSAR 50	Sokkia Japan	50 m / 14 km	3 mm + 5 pp m	1.7
PULSAR 100 _	Sokkia Japan	100 m / 10 km	5 mm + 5 pp m	1.7
PULSAR 50 0	Sokkia Japan	500 m / 15 km	10 mm + 5 pp m	1.85

With the help of light range finders it is possible to measure the distance between two points within the line of sight, therefore the range of action of even laser light range finders on the earth's surface is limited due to the influence of the curvature of the Earth. Radio waves have the ability to bend around obstacles and the earth's surface, so with the help of rangefinders and systems that use the radio wave range of electromagnetic oscillations, longer lines can be measured. However, due to the large divergence of radio waves, the signal reflected from the reflector at the end point will arrive at the starting point weak and it will be very difficult to distinguish it against the background of interference, therefore, an active reflector - repeater is used in radio rangefinders. This repeater is called the slave station.

The principle of operation of radio rangefinders

The radio rangefinder transmitter generates oscillations of the carrier frequency ω_1, which are modulated by oscillations of the scale frequency of the quartz oscillator Ω_1. modulated oscillations are radiated by a parabolic antenna towards the slave station.

Suppose the master station is located at point A , *and* the slave station is located at point B. the current phase of the oscillations of the quartz oscillator at point A is determined by the expression

$$\varphi_1^{A} \backslash u003d\, \Omega_1 t + \varphi_{01}, \quad (76)$$

where φ_{01} is the initial phase.

When passing the distance D to the slave station, the phase of these oscillations at point B will be as follows:

$$\varphi_1^{B} = \Omega_1 (t - \tau_D) + \varphi_{01}, \quad (77)$$

where $\tau_D = D / c$.

The oscillations radiated by the slave station in the direction of the master are modulated by their own crystal oscillator and have a phase

$$\varphi_2^{V} = \Omega_2 t + \varphi_{02}. \quad (78)$$

After passing the distance D to the leading station, the phase of these oscillations at point A will be equal to

$$\varphi_2^{V} = \Omega_2 (t - \tau_D) + \varphi_{02}. \quad (79)$$

Assume that $\Omega_1 > \Omega_2$. After amplitude detection at the master station, a low-frequency (LF) signal with phase

$$\varphi_{\mathrm{I}}^{A} \text{\textbackslash u003d} \varphi_1^{A} - \varphi_2^{A} \text{\textbackslash u003d} (\Omega_1 - \Omega_2) t + \Omega_2 \tau_D + (\varphi_{01} - \varphi_{02}) - \psi_1, \quad (80)$$

where ψ_1 is an additional phase delay in the circuits of the leading station. Similarly, at the slave station, after amplitude detection, an LF signal is formed with a phase

$$\varphi_{\mathrm{V}} = \varphi_1^{V} - \varphi_2^{V} = (\Omega_1 - \Omega_2) t - \Omega_1 \tau_D + (\varphi_{01} - \varphi_{02}) - \psi_2, \quad (81)$$

where ψ_2 is the additional phase delay for the purposes of the slave.

This low-frequency signal, selected at the slave station, is transmitted to the master using additional modulation and, after detection in the frequency detector, is fed to the phase meter. In this case, its phase will be equal to the phase, taking into account the delay obtained as a result of passing the distance D , and the corresponding circuits of both stations:

$$\varphi_{\mathrm{II}}^{A} \text{\textbackslash u003d} (\Omega_1 - \Omega_2) t - \Omega_1 \tau_D + (\varphi_{01} - \varphi_{02}) - \psi_2 - (\Omega_1 - \Omega_2) \tau_D - \psi_3, \quad (82)$$

where ψ_3 - phase delays in the chains of stations.

Consequently, the phase meter receives signals with a phase difference

$$\varphi \text{\textbackslash u003d} \varphi_{\mathrm{I}}^{A} - \varphi_{\mathrm{II}}^{A} \text{\textbackslash u003d} 2\Omega_1 \tau_D - \psi_1 + \psi_2 + \psi_3. \quad (83)$$

This formula was obtained under the assumption that $\Omega_1 > \Omega_2$, i.e. the difference $\Omega_1 - \Omega_2 = \Omega$ *is* positive. The radio range finder on the slave station has two switchable modulation frequencies Ω_2^{+} and Ω_2^{-}, one of which is less and the other is greater than the modulation frequency of the leading station Ω_1 by the same small value Ω. When measuring at the modulation frequency Ω_2^{-} get the phase of the signal

$$\varphi' = -2\Omega_1 \tau_D - \psi_1 + \psi_2 + \psi_3. \quad (84)$$

If we measure the quantities φ, φ' and calculate their half-difference, we get

$$\frac{\varphi - \varphi'}{2} = 2\Omega_1 \tau_D. \quad (85)$$

This expression can be written as

$$\frac{\varphi - \varphi'}{2} = \Omega_1 \tau_{2D}, \ (86)$$

where $\tau_{2D} = 2\tau_D$.

Total travel time of double distance τ_{2D} equals

$$\tau_{2D} = NT + \tau, \qquad (87)$$

where N is an integer number of periods T of frequency Ω_1; τ is the fraction of the period. The phase meter measures only the fraction of the phase cycle - a value not exceeding 2π, i.e. the measurement results give τ - the fractional part of the period. To determine the integer number of periods N in expression (46), measurements are performed at other modulation frequencies.

§ 27. Geometric axes and theodolite conditions.

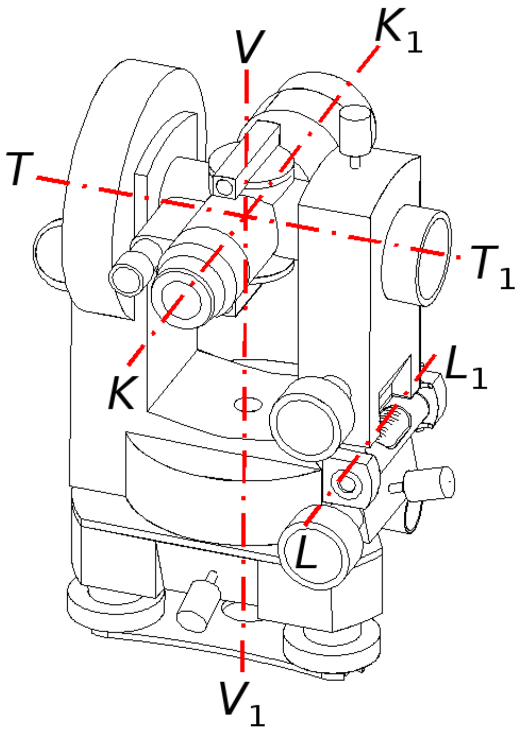


Fig. 39. Theodolite main axes

VV1 is the vertical axis of rotation of the alidade of a horizontal circle; is projected along a plumb line to the top of the measured horizontal angle (centered).

TT1 - horizontal axis of rotation of the telescope.

LL1 - axis of the cylindrical level - tangent to the inner surface of the level ampoule at the midpoint - zero point; horizontal when the level bubble is at zero point.

KK1 - sighting visual tube; fixes the direction to the target; when rotating around the axis, TT1 forms a plane called the collimation plane.

Basic geometric conditions for the ratio of theodolite axes the rotation axis of the alidade VV1 must be perpendicular to the level axis LL1 in any direction;

the axis of rotation of the telescope TT 1 must be perpendicular to the axis of rotation of the alidade VV1;

the sighting axis of the telescope KK1 must be perpendicular to the axis of rotation of the telescope TT1.

The fulfillment of these conditions is regularly checked during the operation of the device (verification) and, if necessary, the axes are brought into the appropriate position (adjustment).

Theodolite checks

To obtain correct measurement results in theodolite, certain geometric conditions must be observed. The presence of these conditions is revealed as a result of verification of the device. If it turns out that one or another geometric condition is not met, the device is adjusted (adjusted) . Perform the following checks.

1. *The axis of the cylindrical level of the horizontal circle must be perpendicular to the vertical axis of the theodolite, i.e. condition must be met VV 1* $\perp$ *LL 1 (* Fig. 39 *).*

By turning the alidade, a cylindrical level is set parallel to the line connecting the two lifting screws, and they bring the level bubble to zero - point. Then rotate the alidade with the level by 180°. If, after turning, the level bubble remains at zero - point or deviates by no more than one division, then the condition is fulfilled. When the bubble is displaced by more than one division, the level adjustment screws move its bubble towards zero - a point by half the deflection arc.

Before performing the following checks, it is necessary to bring the vertical axis of the theodolite to a vertical position. The level is set in the direction of two lifting screws and by rotating them in different directions, the level bubble is brought to zero - point. Then the alidade is rotated by 90° and the bubble is brought to the middle of the alidade with the third lifting screw. These actions are repeated until the bubble remains in the middle of the alidade at any position of the alidade (deviation up to one division is allowed).

2. The *line of sight must be perpendicular to the horizontal axis of the theodolite (pipe rotation axis), i.e. condition RR 1* $\perp$ *TT 1 is required* .

If this condition is met, then when the pipe rotates around the horizontal axis, the sighting axis describes a plane called the collimation one. If the condition is not met, the sighting axis describes not a plane, but two conical surfaces. The angle of deviation of the sighting axis from the perpendicular is called the *collimation error*.

To check this condition, a remote clearly visible object is selected and sighted on it at the position of the vertical circle to the right (KP) of the eyepiece, a reading is taken along the limb E_1, then sighted at the same point when the circle is left (CL), the reading is taken E_2.

The collimation error is calculated by the formula

$$c = \frac{\mathrm{E}_2 - \mathrm{E}_1 \pm 180^\circ}{2}. \quad (88)$$

If it does not exceed double the accuracy of reading along the limb (for the T30 theodolite no more than 2 ′), then the condition is considered to be met. If it exceeds, then adjustment is made. Calculate the correct count using the formula

$$\mathrm{E} = \frac{\mathrm{E}_1 + \mathrm{E}_2 - 180^\circ}{2}. \quad (89)$$

By turning the alidade, this reading is set on the limb of the horizontal circle. In this case, the cross hairs of the grid of threads will shift from the image of the observed point. The vertical adjusting screws of the grid are loosened and, acting with the side adjusting screws, the crosshairs of the grid are moved until it coincides with the observed point. After adjustment, it is recommended to repeat the test.

3. *The horizontal axis must be perpendicular to the vertical axis of the theodolite - condition TT 1* $\perp$ *LL 1.* Compliance with this condition is necessary so that during the measurement of the horizontal angle the collimation plane occupies a vertical position.

To verify this condition, theodolite is installed at a distance of 10 -20 m from the wall of the building. A high point *M is chosen on the wall* and a pipe is pointed at it. Then the point *M is projected* down to a level close to the level of the horizontal axis of the theodolite, and the projection of the point *M ′* is marked on the wall. At the second position of the vertical circle, the same actions are repeated and a second projection of the point M″ is obtained. Mismatch of points *M ′* and M″ within the strip of the bisector is allowed. Adjustment is carried out in the factory or in specialized workshops. It should be borne in mind that the average of readings along the limb, taken when aiming at a point at two positions of the tube (KP and KL), is free from the influence of the tilt of the axis of rotation of the tube.

4. *Verification of the grid of threads - the vertical stroke of the grid of threads must be located in the collimation plane of the tube, or otherwise, the horizontal stroke of the grid of threads must be perpendicular to the axis of rotation of the theodolite.*,

Point the crosshairs of the grid of threads at a clearly visible point. turning alidade with a lead screw, observe the position of the point. If the image of the point remains on the middle horizontal stroke of the grid, the condition is met. For adjustment, the grid screws are unfastened and rotated accordingly, after which the screws are fixed, and the check is repeated.

If the adjustment (rotation) of the grid was carried out, then after this verification it is necessary to repeat the verification of the perpendicularity of the sighting axis to the horizontal axis of the theodolite.

§ 28. Theodolite and its device. Reading device. Limb and alidade. Reading microscope. Net of threads.

In geodesy, instruments such as theodolites. With their help, during geodetic work, specialists measure horizontal, vertical angles and distances.

The device is based on a sighting tube, as well as reference circles (horizontal and vertical). The tube has a certain magnification and works on the principle of a telescope. It is mounted on two columns, which, in turn, are fixed on a special base. It is mounted on a stand called a tribrach.

Theodolite includes the following components:

- limb is a goniometric circle having divisions from 0 to 360 degrees, during measurements it plays the role of a working measure;
- alidada - a movable part of the structure that carries the reference system in a circle and holds the sighting tube;

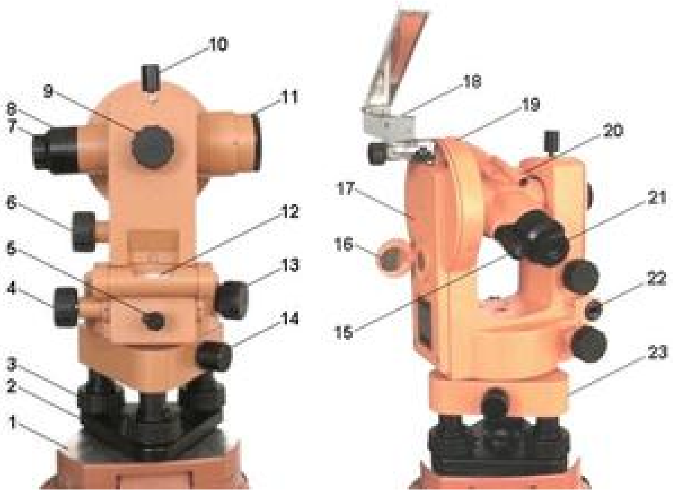


Fig.40. View of theodolite, its components and parts

- spotting scope - it is attached with stands to the alidade part;
- axial system - helps to move the alidade part and limbus around the axis;
- vertical circle - helps to measure vertical angles;

- stand equipped with several lifting screws;
- leading and clamping screws of moving parts. Leading ones are also called micrometric, and clamping - fixing;
- a tripod and a hook for a plumb line, along with a platform for a stand and a dead screw;
- circle adjustment screw;
- levels for vertical and horizontal circle;
- focus screw;
- microscopic eyepiece for a reading device.

Devices differ in the type of accuracy, areas of use and design features. In addition, each classification determines what theodolite is intended for and in what works it will be more useful.

In terms of accuracy, they are:

- high-precision - the error is less than 1.5 '';
- accurate - the error rate ranges from 1.5 to 10 '';
- optical (technical) - an error of 10 '' and above.

According to the scope of use, the structures are divided into:

- autocollimation;
- gyroscopic;
- mine surveying;
- compass;
- geodetic;
- astronomical.

According to the design features of the optical system, pipes come with reverse or direct images.

It is worth mentioning the differences between a theodolite and a level. The difference lies in the fact that the theodolite can perform not only horizontal leveling, but also measure vertical angles.

Theodolites have changed over time . The very first samples had a ruler on the tip of the needle in the center of the goniometric circle, which rotated freely on it. There were cutouts on the ruler, and there were also stretched threads on them, acting as reference indices. And the center of the goniometric circle was set at the top of the corner and firmly fixed.

When turning the ruler, it was combined with the first side of the angle, then a reading was taken on the scale of the goniometric circle. And then the ruler was combined with the other side of the corner, and the second count was taken. The difference between the two values corresponds to the value of the angle. In order to align the ruler with different parts of the angle, simple sights were used.

Nowadays, the design of the device has improved significantly . So, to align the ruler with the sides of the corner, a pipe is used that moves in height and azimuth. For counting, a special device is also used, its modern design, which, unlike its "ancestors", is covered with a protective metal casing.

An axial system is used to ensure smooth rotation of the moving elements, while the movements themselves are regulated by means of guide and clamping screws. Theodolite is installed on the ground on a tripod, and the center is aligned with a plumb line by means of a plumb line or an optical plummet.

The sides of the angle to be measured are projected onto the plane of the circle using a vertical moving plane (collimation). It is formed through the sighting axis of the pipe when it rotates around its axis. The line of sight is an imaginary line that passes through the center of the reticle and the optical center of the lens.

Electronic theodolites are modern instruments for measuring angles. Their use eliminates errors when taking readings, since the values are displayed on a special screen in the form of numbers. The display is carried out due to the fact that special sensors are built into the horizontal and vertical circles.

Working with such a device is much easier than with a conventional one. Some electronic models are equipped with additional functions for automating work. However, simple optical designs are still preferable in some situations:

- they do not need recharging;
- able to work stably even in extreme conditions.

But electronic devices cannot be used at low temperatures (less than 30 degrees below zero).

What a theodolite is for is determined by its accuracy. **The main areas of use of the device are:**

geodetic thickening networks;
triangulation;
polygonometry;
applied geodesy; structural elements of machines and mechanisms);
construction of industrial facilities and not only.

- In the line microscope of the T30 theodolite, a stroke is visible in the middle of the field of view, relative to which the reading is carried out along the limb (Fig. 41, *a*). Before reading along the limb, it is necessary to determine the division value of the limb. In the T30 theodolite, the value of the division of the limb is 10 minutes of arc, since the degree is divided into six parts. The number of minutes is estimated by eye in tenths of the price of the division of the limb. The reading accuracy is 1'.
- In a scale microscope, a scale is visible in the field of view, the size of which corresponds to the division value of the limb (Fig. 41, *b* , *c*). For a theodolite of technical accuracy, the scale size and the value of the division of the limb are 60'. The scale is divided into twelve parts and the price of its division is 5 arc minutes. If there is no minus sign before the number of degrees, the reading is made on a scale from 0 to 6 in the direction from left to right (Fig. 41, *b*). If there is a minus sign before the number of degrees, then the minutes are counted on the scale of the vertical circle from -0 to -6 in the direction from right to left (Fig. 41, *c*). Tenths of the scale division value are taken by eye with an accuracy of 30".

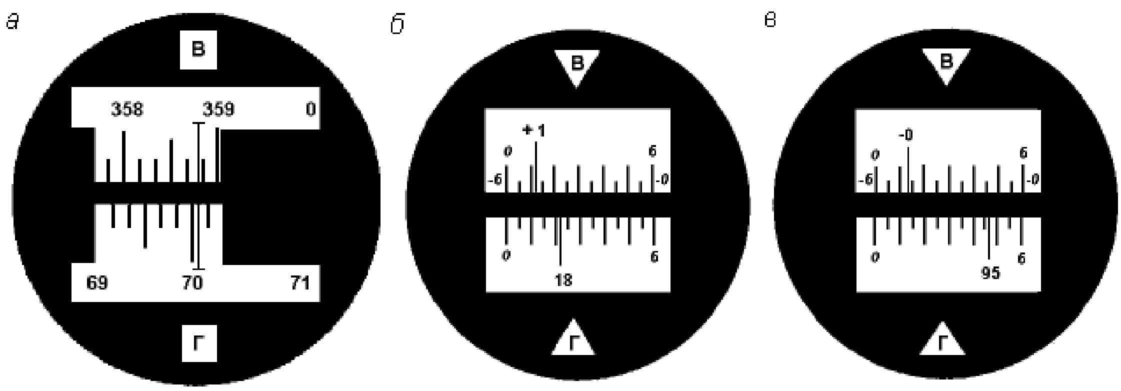


Fig.41. The field of view of reading devices: *a* – a streak microscope with readings along the vertical circle 358°48', along the horizontal 70°04'; *b* – scale microscope with readings: along

the vertical circle 1°11.5', along the horizontal 18°22'; *c* - in a vertical circle - minus 0°46.5', in a horizontal circle - 95°47'

In order for the theodolite to provide undistorted measurement results, it must satisfy the appropriate geometric and optical-mechanical conditions. The actions associated with checking these conditions are called ***verifications*** . Theodolite checks are carried out in accordance with the passport-instruction attached to the device, or the instruction for carrying out technological verification of geodetic instruments.

If any condition is not met, the device is adjusted with the help of corrective ***screws*** .

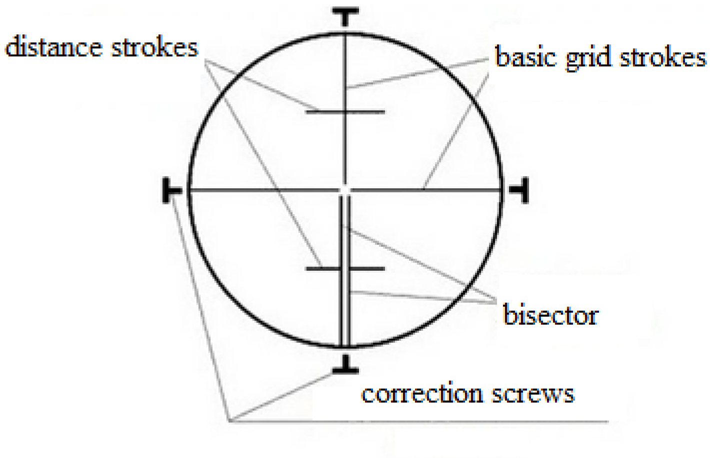


Fig.42. Net of threads

Chapter 7
Checking and adjusting theodolite

§ 29. Checking and adjusting theodolite . § 30. The procedure for performing verifications.

Before starting work with the theodolite, an external inspection checks its stability on a tripod, the smoothness of the lifting and pointing screws, as well as the strength of fixing the rotating parts with fixing screws.

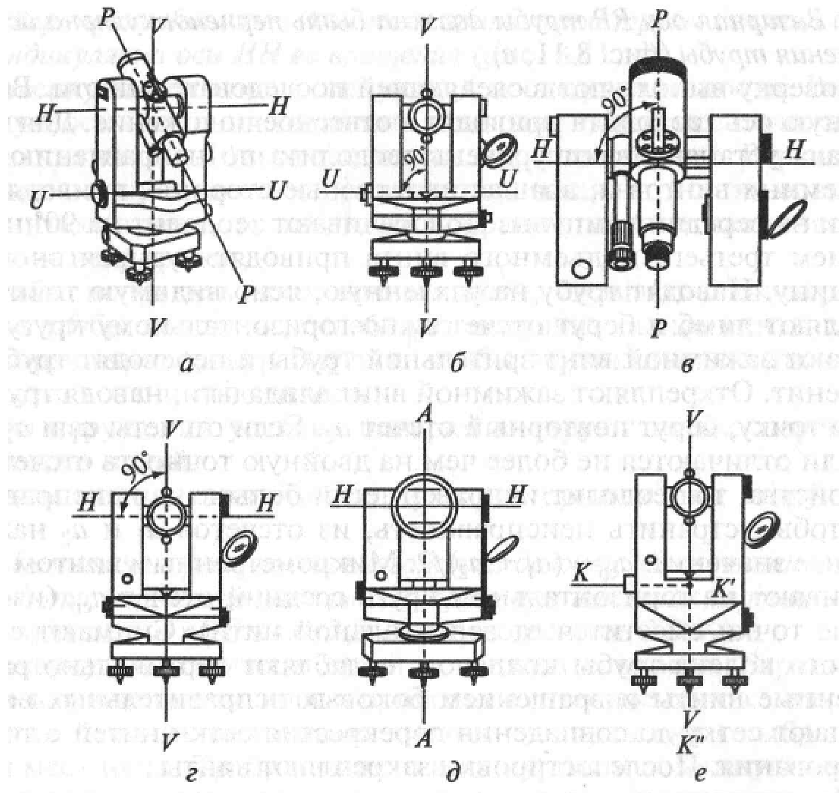


Fig.43. Schemes *(a, b, c, d, e)* of theodolite geometric axes.

If the theodolite is received from the factory, after repair or from another specialist, then checks are performed before the theodolite is put into operation. In the process of verification, they make sure that the relative position of the axes of the device is correct (Fig. 43, *a).*

1. The axis *UU* of the cylindrical level of the horizontal circle must be perpendicular to the axis *VV* of rotation of the device (Fig. 43, b).

Verification is performed in the following sequence. The theodolite is mounted on a tripod so that the level is located in the direction of any two lifting screws and, by rotating them in different directions, the level bubble is brought to zero point, then the horizontal circle of the theodolite is rotated 180 °. If the bubble remains in the middle or deviates by no more than one division, then the level is correct, if more than one division, it is faulty.

To eliminate the malfunction, the bubble is moved by the corrective level screws to the zero point by one half of the deflection arc, and by the lifting screws - by the second.

After checking, make sure that the theodolite maintains its working position. To do this, the horizontal circle is rotated by 90°, the bubble of the cylindrical level is brought to the middle and the horizontal circle is rotated in an arbitrary direction. If, at different positions of the circle relative to the lifting screws, the bubble remains in the middle, then the verification is considered completed.

2. The sighting axis of the *PP* pipe must be perpendicular to the axis of the *LV* rotation of the pipe (Fig. 43, c).

Verification is performed in the following sequence. The vertical axis of the theodolite is brought into a vertical position. To do this, first set the level of the theodolite in the direction of two lifting screws and, rotating them in different directions, bring the bubble to the middle of the ampoule. Turn the theodolite 90° and turn the third lifting screw to bring the bubble back to the middle. Point the pipe at a remote, clearly visible point, fix the limb and take a reading a_1 in a horizontal circle. The clamping screw of the telescope is released and the telescope is moved through the zenith. The clamping screw of the alidade is unfastened and, pointing the pipe at the same point, take a second reading a_2. If the readings a_1 and a_2 are equal or differ by no more than double the accuracy of the reading device, then the theodolite is operational, if more, it is faulty.

To eliminate the malfunction, from the readings a_1 and a_2 find the average value: $a_{cf} = (a_1 + a_2)/2$.

With a micrometer screw, the average reading a_{cp} is set on a horizontal circle (the image of the point will shift from the vertical thread). The cap is removed from the ocular knee of the tube, the vertically located screws are loosened, and the grid is displaced by rotating the side corrective screws until the crosshairs of the grid of threads coincide with the sighting point. After adjustment, fix the screws.

It is possible to measure the angle even if the ratio of the axes is violated. In this case, readings are taken at two positions of the pipe - left (L) and right (R) and the average is determined from these readings.

3. Axis *HH* of rotation of the pipe must be perpendicular to the axis *VV* of rotation of the device (Fig.43, d).

Verification is performed in the following sequence. Theodolite is installed at a distance of 10 ... 15 m from the wall of the building. The vertical axis of rotation is brought to a vertical position. The pipe is pointed at a point high on the building, and a horizontal circle is fixed. The pipe is smoothly lowered to a horizontal position. The projection of the point is marked on the wall. Transfer the pipe through the zenith, lower the fixing screw of the alidade and point again at the same point. A point is projected onto the same level and fixed. If the projections of the point match, then the theodolite is working, if they do not match, it is faulty.

The conditions for this verification are guaranteed by the manufacturer. If the conditions are violated, the device is sent to the workshop for repair.

When working with a broken ratio of the axes, measurements are made only at two positions of the circle. When the pipe is raised up to 30 ° and the distance to the projected point is up to 20 m, mismatch of projections up to 30 mm is allowed; the average of the two inductions is taken as the final result.

4. The vertical thread *AA* of the reticle of the telescope must be perpendicular to the axis *NN* of its rotation (Fig. 43, e).

Verification is performed in the following sequence. The vertical axis of rotation of the theodolite lead to a vertical position. At a distance of 8 ... 10 m from the theodolite, a plumb line is fixed. The vertical thread is directed to a plumb line. If the vertical thread of the grid coincides with the thread of the plumb line, then the theodolite is working, if it deviates from the plumb line, it is faulty.

To correct the ratio of the axes, remove the cap from the ocular elbow of the tube, loosen the correction screws of the grid and turn the diaphragm so that the vertical thread of the grid is aligned with the plumb line. If the verification conditions are violated, only the cross hairs of the grid of threads are sighted.

After performing this verification, a second verification is repeated.

5. The sighting axes of optical sights must be parallel to the sighting axis of the telescope.

The fulfillment of this condition is necessary for the convenience of working with the theodolite and reducing the time of sighting at the observed objects. Verification is performed as follows. They sight with a spotting scope at a clear distant point, which is also clearly visible to the naked eye. Next, consider this point without a pipe with one eye and at the same time consider the crosshairs of the sight with the other eye. If the crosshair image of the reticle is aligned with the image of the observed point, then the condition is met.

If the condition is not met, then loosen the screws securing the sight to the body of the telescope, and turn it in the desired direction. Then the screws are tightened.

. § 31. Measurement of the horizontal angle.

The measurement begins with the fact that the theodolite is fixed on a tripod and centered over a point using a plumb line (filament or optical). Lifting screws lead the vertical axis of the theodolite to a vertical position, fix the limb. Prepare a tube for observations. Then the pipe is pointed at the points (the alidade rotates), counted along the limb, and the angle value is calculated. Consider two ways to measure horizontal angles.

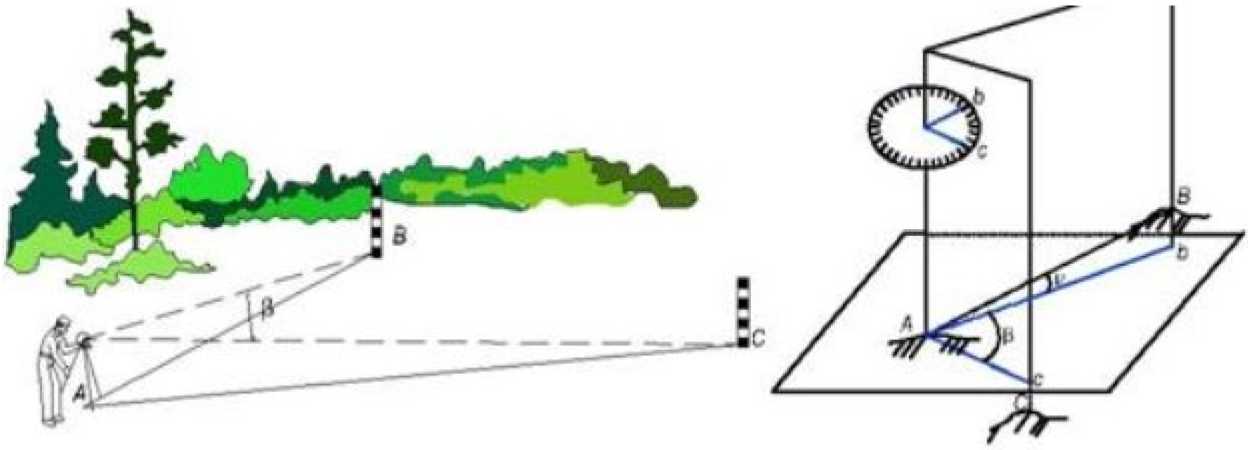


Fig.44. Measurement of horizontal angles by the method of receptions

The method of receptions, the first half-reception. With a fixed limb, turning the alidade, they sight at point *A* (Fig. 44) and receive a reading along limb a_1, observe at point *c* and receive a reading *from* $_1$. calculate the value of the angle β from the first half-step $\beta = c_1 - a_1$.

Second half. They transfer the pipe through the zenith and, turning the alidade, first observe point *C*, get a reading *from* $_2$, and then to point *A*, get a reading a_2. angle value from the second half-receive

$$\beta' = c_2 - a_2. \quad (90)$$

If the discrepancy between β and β' does not exceed double precision, calculate the average value of the measured angle

$$\beta_{cf} = \frac{\beta + \beta'}{2}. \quad (91)$$

Otherwise, the measurements are repeated, having previously turned and fixed the limb in a different position.

The method of circular receptions

This method is used when there are more than three observed directions. To facilitate calculations, the theodolite limb is set so that the reading in the first direction a_1 is close to zero, then, with the limb fixed, they sight at the points and record the readings a_1, a_2, ..., a_n, and at the end for control again point to the first point.

In the second half-reception, the pipe is transferred through the zenith and sighted in all directions in the reverse order - counterclockwise. The values of the directions in the half-points are calculated relative to the first direction, taking the first one as zero (the reading a_1 is subtracted from all readings). Then the average values of the directions from the half-points are calculated. Differences in the values of directions in half-receives are allowed within the double precision of counting.

§ 32. Measurement of a vertical angle

A vertical angle is a flat angle lying in a vertical plane. Vertical angles include the angle of inclination and the zenith distance. The angle between the horizontal plane and the direction of the terrain line is called the angle of inclination and is denoted by the letter v. Tilt angles are positive and negative.

The vertical angle is measured using a vertical circle fixed on the axis of rotation of the telescope. The zero reading of the limb of the vertical circle, rigidly fastened to the axis of the pipe, is set parallel to the sighting axis of the pipe. If the geometric conditions of the vertical circle are violated, an instrumental error occurs, called the zero point of the vertical circle (see theodolite verification), and the vertical angles are not measured from the zero diameter of the limb.

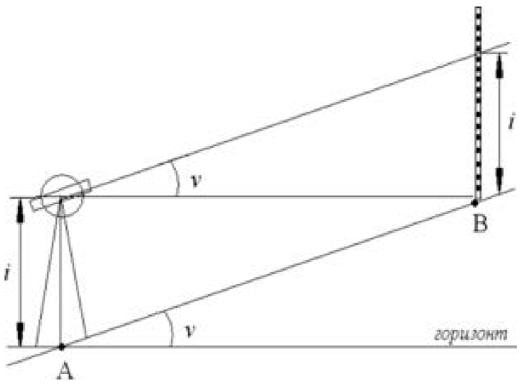


Fig. 45. Scheme for measuring the vertical angle

Measurements begin with the installation of the device (theodolite) at point A. Bring it to the working position and use a tape measure to measure the height of the device i (the distance from the axis of rotation of the pipe to the point above which the device is installed). At point B, a rail is vertically installed, on which the height of the device i is marked. They sight at the height of the instrument and measure the vertical angle, which will be equal to the angle of the terrain.

The method of measurement depends on the design and digitization of the vertical circle of the theodolite.

First way. If the vertical circle does not have a level during alidade, then after bringing the device into working position, sight on the point to be determined. For example, with a CL, the alidades of a vertical circle lead to the zero-point level with a vertical circle and take a reading along the limb of the vertical circle. The pipe is transferred through the zenith and the actions are repeated at a different position of the vertical circle. Calculate the value of the zero point (MO) of the vertical circle (see formula (6)) and the vertical angle:

v = KJl-M0, y = M0-KP. (92)

The control of the correctness of measurements is the constancy of MO, the fluctuations of which can be within the double accuracy of the device (M0=const, AM0<2t').

The second method is used if the alidade of the vertical circle does not have a level, and its functions are performed by the level with the alidade of the horizontal circle (T3O, 2T30). The device is brought into working position, preliminarily sighted at the point to be determined; the lifting screw located closest to the sighting axis, zero-point the level bubble at the GK; produce accurate sighting and take a reading on the scale of the vertical circle. The action is repeated with a different position of the vertical circle. The vertical angle and MO are calculated using formulas (6) and (7). Control is the constancy of MO.

The third method can be used if the alidade of the vertical circle does not have a level and a compensator is used instead (the alidade automatically becomes horizontal). The order of measurements: the device is brought into working position, sighted at the point to be determined and a reading is taken on the scale of the vertical circle. The telescope is transferred through the zenith and the actions are repeated. Calculate the vertical angle and MO. Control is the constancy of MO.

§ 33. Place of zero (MO)

In this case, the line, relative to which the reference is taken, will occupy a certain, but not necessarily horizontal, position. It would seem that when the level axis is brought to a horizontal position, the reading along the vertical level should be equal to zero, since the vertical angle is measured from the horizontal plane. However, due to various reasons, this does not happen, and the reading along the vertical circle with the horizontal position of the sighting axis and the level axis differs from zero. Just as in a number of physical instruments, the so-called "zero reading" can take place - a position in which the arrow of the instrument, even before the start of measurements, is set not at zero, but at some other figure. This "zero count" in geodesy is called the zero point (MO). It must be taken into account as a reading correction when measuring vertical angles. So, for example, for the T30 theodolite, the angle of inclination of the sighting axis of the pipe and the reading L in a vertical circle at CL are related (Fig. 45) by the relation

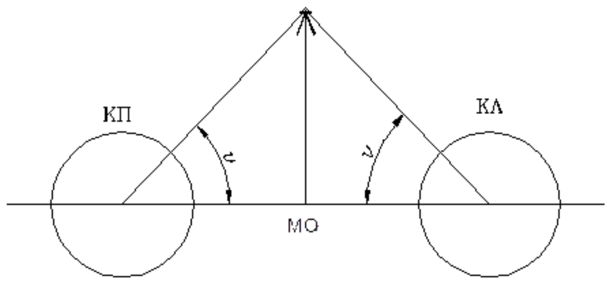


Fig. 46. Scheme for checking the place of zero

v \u003d L - MO. (93)

The relationship between the angle of inclination *v* and reading P at the CP will be as follows for the T30 theodolite:

v \ u003d 360 ° - (P + 180 °) + MO \u003d MO - (P + 180 °). (94)

In the T30 theodolite, the divisions on the vertical circle increase counterclockwise, so 180 ° are added to the reading at the CP.

Solving these two equations for *v* and MO, we get:

$$v = \frac{Л - П - 180^{\circ}}{2} \qquad (9\ 5)$$

and

$$МО = \frac{Л + П + 180^{\circ}}{2}. \qquad (\ 96)$$

It follows from formula (95) that at MO = 0 the readings along the vertical circle correspond to the measured tilt angles. Therefore, the place of zero is a reading along the vertical circle of the theodolite with a horizontal position of the sighting axis of the telescope and the level axis.

Depending on the digitization of the vertical circle of theodolites of different models, formulas (93) and (94) change somewhat; they are given in the passport of each device.

When sighting at points with different heights, the zero point must remain constant. The constancy of the MO serves as a control for the correctness of the measurements.

The verification of the constancy of MO is carried out as follows. They sight at several local objects at different distances from the device at the CP and CL, and according to the readings obtained in the vertical circle, the values of the zero point are calculated using the formula (96). The results obtained must be the same within double the accuracy of the reading device.

Calculations of tilt angles using formula (96) can be simplified if the value of the zero place is equal to or close to 0.

To bring the place of zero to zero in the theodolite T30 after determining the value of the place of zero at the position of KL, the value of the angle of inclination calculated by the formula (96) is set on the vertical circle. As a result, the horizontal thread of the grid will shift from the observed object. Acting with vertical corrective screws of the mesh of threads, the middle thread of the mesh is combined with the image of the observed object. After correcting the value of the zero place, it is determined again and, if necessary, corrected again.

§ 34. Electronic total stations.

Currently, in the practice of geodetic work, devices that combine a digital theodolite, an electronic range finder, a processor and software for processing measurement results are widely used. These devices also have a storage unit that accumulates all the information received.

The latest generation total station has a built-in laser non-reflective range finder that allows you to determine the coordinates of any point in the terrain (or object) on which the pipe is pointed. When working with a reflector, servo drives are used to facilitate aiming at the installed reflector, and the process of aiming at the reflector is carried out automatically. Thus, the presence of an observer at the device becomes optional.

The surveyor installs the total station at the station, brings it into working position. Then enters into the processor the coordinates of the standing point, the value of the directional angle when the pipe is directed to the neighboring station and the device is ready for operation. In another version, the device is installed in a place convenient for work, the pipe is directed sequentially to three points, the coordinates of which are known and entered into the processor memory, and the directions and distances to these points are measured. The position of the standing point is determined by the reverse of the angular and linear serifs from these three points.

A topographic survey specialist installs a reflector at a point in the area. The tacheometer finds the reflector, determines its coordinates (point coordinates) and, at the signal of the topographer, remembers the point number, coordinates and its code (designation, for example, the corner of a house, a sewer hatch, etc.)

If necessary, the coordinates of the surveyed point can be obtained via radio channel and at the reflector installation point. To do this, a remote control with a display is fixed on the reflector rod.

Electronic tacheometers are universal devices, they can be used when creating a planned-altitude survey justification, in the process of tacheometric survey, as well as when performing layout work.

Laser total stations

A new class of tacheometers has taken shape - laser tacheometers.

It is more correct to call them total stations with a non-reflective laser rangefinder. With the advent of the laser tape measure, the idea arose to combine it with a total station. At the first stage, the laser tape measure was fixed on the theodolite tube using a special bracket. Two solutions are possible depending on the method of fixing. The laser beam was directed parallel to the sighting axis in the vertical plane or parallel to the sighting axis in the plane perpendicular to the axis of rotation of the pipe. Such solutions were forced and created certain difficulties associated with the need to reduce the measurement results, as well as the fact that when performing measurements using the reflectorless method, errors could occur due to the separation of the sighting axis and the axis of the laser beam.

The developed laser total stations have a laser non-reflective rangefinder, the laser beam of which is aligned with the sighting axis of the pipe. Work with such a total station is possible both in reflectorless mode and with a reflector. In the latter case, the range of the device increases significantly.

Table 14 **Total Stations**

Technical characteristics	SET 4110R SOKKIA Japan	ELTA [R] C Zeiss Germany	NIKON DTM-450 Japan	Trimble 5601 USA
Angular error, ″	5	2/3	5	one
Tilt compensator axes of rotation	2 - axial	2 - axial	1 - axial	-
Working range	3′	5'	3′	-
Linear error measurements mm + mm/km	5+3	2+2	2+2	2+2
Measurement time C	-	-	-	-
range with one reflector, m	1000	2500	2700	2500
Maximum, m	4000	3000	4400	3500
Reflectorless, m	100	-	-	600
Keyboard (number of keys)	28	28	fifteen	27
Device weight, kg	5.6	6.2	6.2	7.5

Two models are produced: AUTLOCK - automatic targeting and tracking of the reflector up to 2200 m; ROBOTIC - robotization of measurements alone (connection of the reflector installation point with the tacheometer via radio channel).

Chapter 8
Geodetic tasks

§ 35. Direct geodesic problem.

Determining the (rectangular) coordinates of a point from the known coordinates of another point, the directional angle, and the horizontal distance between them.

In geodesy, it is often necessary to transfer coordinates from one point to another. For example, knowing the initial coordinates of point A (Fig. 47), the horizontal distance SAB from it to point B, and the direction of the line connecting both points (directional angle αAB or rhumb rAB), you can determine the coordinates of point B. In this formulation, the transfer of coordinates is called a direct geodesic task.

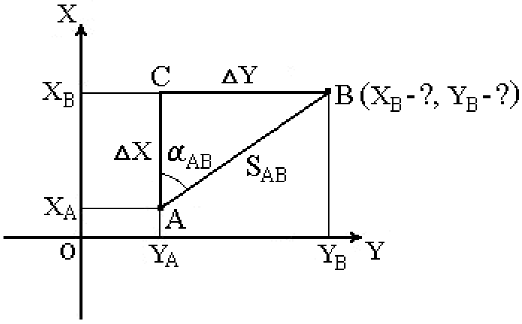


Fig. 47. Direct geodesic problem

For points located on a spheroid, the solution of this problem presents significant difficulties. For points on a plane, it is solved as follows.

Given: Point A(XA, YA), SAB and αAB.

Find: point B(XB, YB).

Directly from the figure we have:

$$\Delta X = XB - XA; (97)$$

$$\Delta Y=YB-YA. (98)$$

The differences ΔX and ΔY of the coordinates of the points of the next and previous ones are called coordinate increments. They are projections of the segment AB on the corresponding coordinate axes. We find their values from the rectangular rectangle ABC:

$$\Delta X = SAB \cos \alpha AB ; (99)$$

$$\Delta Y = SAB \sin \alpha AB. (100)$$

When calculating, the signs of the increments will depend on the direction (α_{AB}).

§ 36. Inverse geodesic problem.

The inverse geodesic problem is that, with known coordinates of points **A** (X_A, Y_A) and **B** (X_B, Y_B) , it is necessary to find the length S_{AB} and the direction of the line **AB** *:* rhumb r_{AB} and directional angle α_{AB} (Fig. 48).

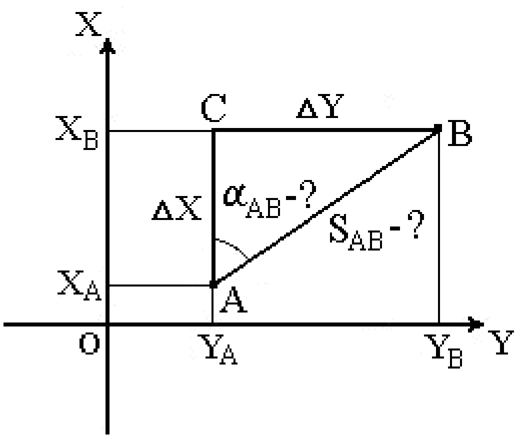


Fig. 48. Inverse geodesic problem

This problem is solved in the following way.

First, we find the increments of coordinates:

$$\Delta X = X_B - X_A ; (101)$$

$$\Delta Y \text{\u003d} Y_B - Y_A. \quad (102)$$

The value of the angle r_{AB} is determined from the relation

$$\frac{\Delta Y}{\Delta X} = \tan r \quad (103)$$

By signs of increments of coordinates, a quarter is calculated in which the rhumb is located, and its name. Using the relationship between directional angles and points, we find α_{AB}.

For control, the distance S_{AB} is calculated twice by the formulas:

$$S_{AB} = \frac{\Delta X}{\cos \alpha_{AB}} = \frac{\Delta Y}{\sin\alpha_{AB}} = \Delta X \sec \alpha_{AB} = \Delta Y \operatorname{cosec} \alpha_{AB} \quad (104)$$

$$S_{AB} = \frac{\Delta X}{\cos r_{AB}} = \frac{\Delta Y}{\sin r_{AB}} = \Delta X \sec r_{AB} = \Delta Y \operatorname{cosec} r_{AB} \quad (105)$$

The distance S_{AB} can also be determined by the formula:

$$S_{AB} = \sqrt{\Delta X^2 + \Delta Y^2}. \quad (106)$$

§ 37. Determination of signs of increments of coordinates and trigonometric functions.

Table 15.Signs of increments of coordinates δx and δy

increments coordinates	Quarter of the circle into which the line is directed			
	I (CB)	II (SE)	III (SW)	IV (NW)
ΔX	+	-	-	+
ΔY	+	+	-	-

Using the relationship between directional angles and axial points (Fig. 49), we find α_{AB}.

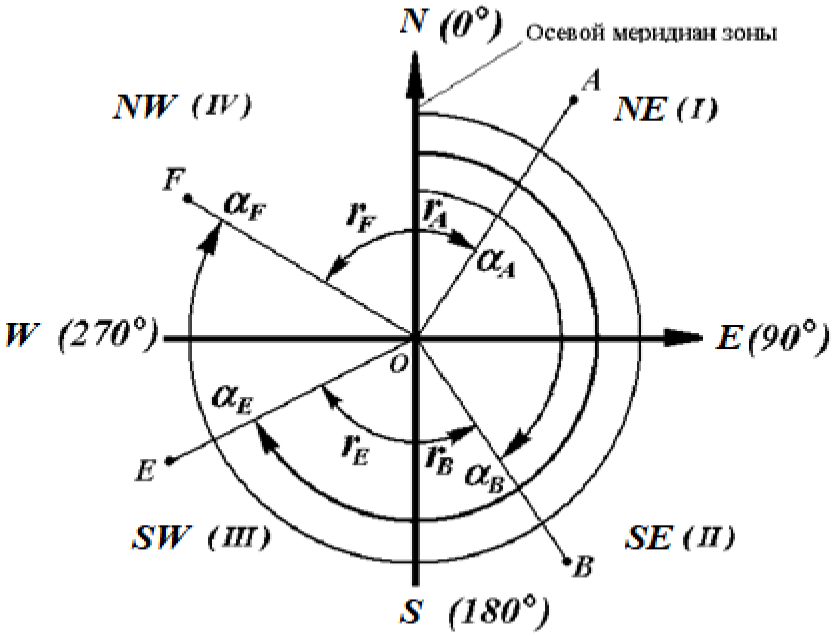


Fig. 49. Axial points and directional angles

The relationship between directional angles and rhumbs is determined for quarters by the following formulas:

I quarter (CB) $r = \alpha$, (107)
II quarter (SE) $r = 180° - \alpha$, (108)
III quarter (SW) $r = \alpha - 180°$, (109)
IV quarter (NW) r \u003d 360 ° - α. (110)

The distance S_{AB} is determined by the formula

$$S_{AB} = \sqrt{\Delta X^2 + \Delta Y^2}. \quad (111)$$

To control the distance S_{AB} is calculated twice by the formulas:

$$S_{AB} = \frac{\Delta X}{\cos \alpha_{AB}} = \frac{\Delta Y}{\sin \alpha_{AB}} = \Delta X \sec \alpha_{AB} = \Delta Y \operatorname{cosec} \alpha_{AB}, \quad (112)$$

$$S_{AB} = \frac{\Delta X}{\cos r_{AB}} = \frac{\Delta Y}{\sin r_{AB}} = \Delta X \sec r_{AB} = \Delta Y \operatorname{cosec} r_{AB}. \quad (113)$$

Thus, you can find the coordinates of any number of points according to the rule: the coordinates of the next point are equal to the coordinates of the previous point, plus the corresponding increments.

§ 38. Methods for determining the position of points on the ground.

Rectangular coordinate method (perpendicular method).

The side of the stroke closest to the contour is taken as the abscissa axis, point A is taken as the origin. The position of each point is determined by rectangular coordinates X and Y .

Abscissas are usually measured using a measuring tape, and ordinates are usually measured using a tape measure. The method of perpendiculars is used mainly when shooting elongated contours .

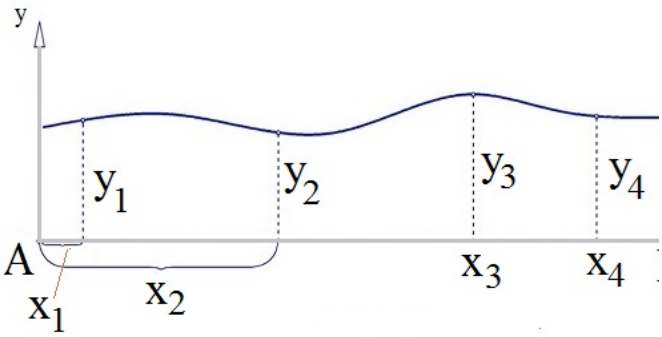


Fig.50. Rectangular coordinate method (perpendicular method).

Polar coordinate method (polar method).

In this case, the side of the theodolite traverse closest to the contour is taken as the polar axis, the beginning of the line is taken as the pole. The position of points 1, 2, 3 is determined by the polar angles ß $_1$, ß $_2$, ß $_3$; radius - vectors d $_1$, d $_2$, d $_3$.

Polar angles are measured using a theodolite in one half-step, with the limb oriented along the sides of the stroke, the sides are measured using a filament rangefinder . When shooting especially important contours, use tape .

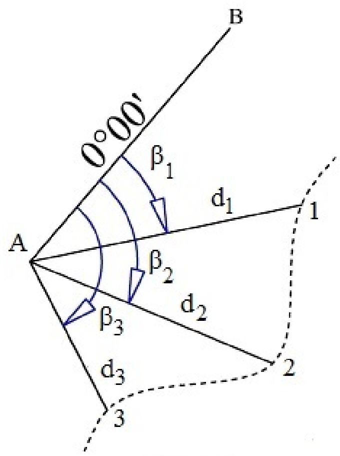


Fig.51. Polar coordinate method (polar method).

Linear serif method

Triangles try to make close to equilateral. Linear serif is often used when shooting buildings. In this case, the distances are measured with a tape or tape measure.

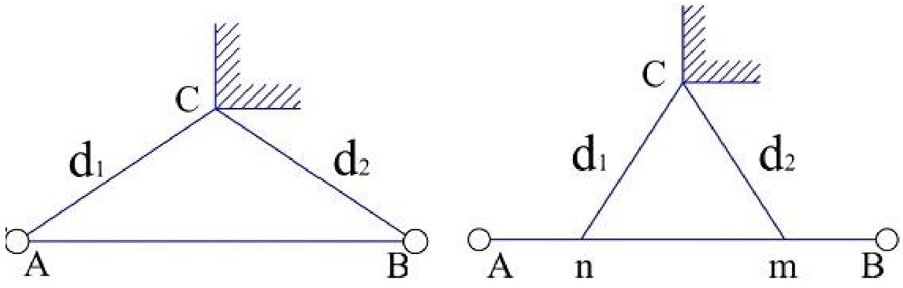


Fig.52. Linear serif method

Angled serif method

The method of angular serifs is used in cases where it is not possible to determine the position of a point using linear measurements.

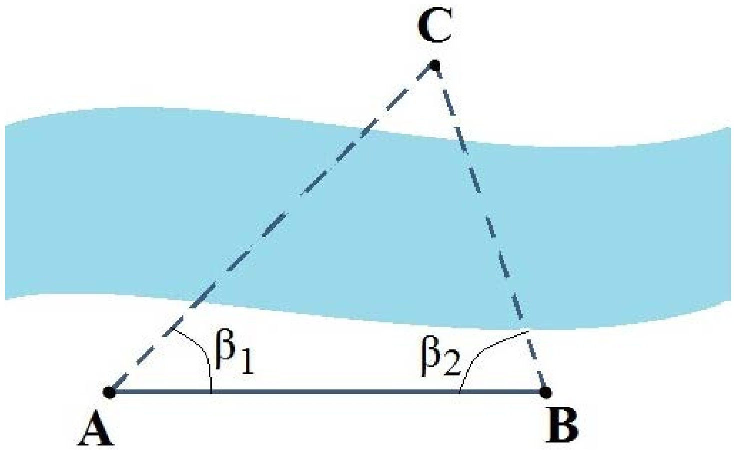


Fig.53. Angled serif method

Slot method. The position of the point P is determined by the distance 2-P along the line 2-E . The position of the leading line is determined by the distance 4-E .

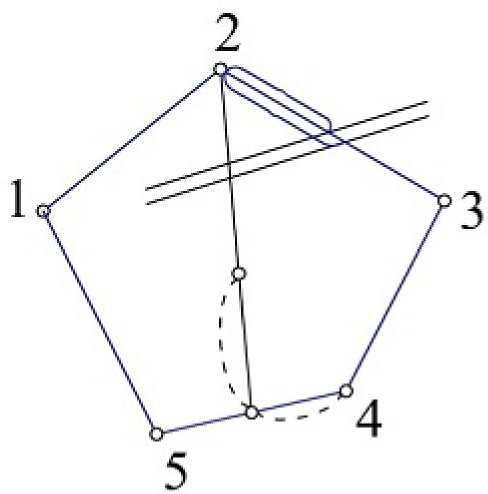


Fig.54. Slot method.

When shooting a situation, an outline is drawn up .

§ 39. Calculation of the directional angle.

The following scheme for their calculation is proposed. There are 2 reference points for polygonometry PP 87, PP 88. The directional angle α of this line is the initial one and is given separately for each option. Having the initial directional angle and a log of measuring horizontal angles of both the main polygon and the reference traverse, you can calculate all the directional angles of the polygon according to the following scheme, Figure 55.

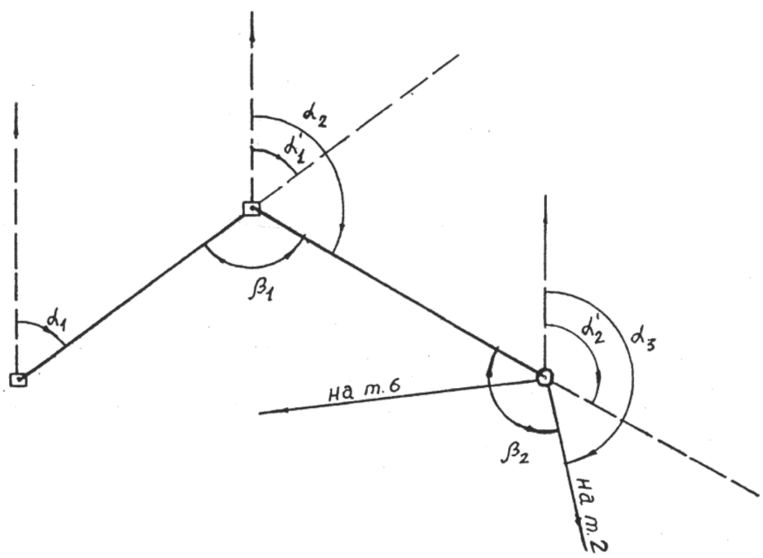
You can find angles using the formula:

$$\alpha_{i+1} = \alpha_1 + 180^0 - \beta_{i\ ИСПРАВ}, \quad (114)$$

where α_{i+1} is the directional angle of the subsequent side of the theodolite traverse;

α_1 - directional angle of the previous side of the theodolite traverse;

$\beta_{i\ ИСПРАВ}$ - the right angle along the course between the named sides of the theodolite traverse.



α 1	- initial directional angle ;
β 1 , β 2	- coal binding (right along the way);
α 2	- directional binding angle;
α 3	- directional angle of the line 1-2 of the main polygon.

Fig.55. Scheme for calculating directional angles

The control for calculating directional angles is to obtain the directional angle of the line between vertices 1 and 2 of the main polygon, which is calculated twice, at the beginning and at the end of the list of coordinates with the same result. After calculating the directional angles, they are converted to rhumbs. The translation of directional angles into rhumbs is given in table16.

Table 16. Translation of directional angles into rhumbs

Quarter	Sign	Relationship between rhumbs

	x	y	and directional angles
I SW	+	+	r = α
II SE	-	+	r = 180˚ - α
III SW	-	-	r = α - 180˚
IV NW	+	-	r = 360˚ - α

Chapter 9
Geodetic networks

§ 40. History, structures and legal framework of the state cartographic service

To ensure the possibility of mapping the country (including geological), solving scientific and practical problems, a network of points has been placed on the territory of Uzbekistan, forming a system of points reliably fixed on the ground, the coordinates of which are obtained with high accuracy in a single system. It constitutes *the state geodetic reference network* , implemented on the basis of the following basic principles: continuity; the required density of points; accuracy sufficient to solve scientific and practical problems.

The state geodetic network is divided into planned and high-rise (leveling). The first one is used to determine the planned coordinates (x , y), the second - to determine the absolute heights. The basic principle of building networks is from the general to the particular - from large high-precision to smaller and less accurate.

In accordance with this, geodetic networks are subdivided into state networks, concentration networks, and survey networks.

Existing planned and high-altitude geodetic base and cartographic security of the territory of the Republic of Uzbekistan . Old State Geodetic Network • The Old State Geodetic Network (planned basis) of the Republic of Uzbekistan is a fragment of the general GGS on the territory of the Commonwealth of Independent States (CIS) and consists of the following networks: triangulation class 2; • Geodetic Condensation Networks (GSN) of 3rd and 4th classes. • The density of GHS points of 1, 2, 3 and 4 classes (excluding urban networks) is 1 point per 32 sq. km. km. • Heights of GGS points are determined from geometric or trigonometric leveling. • ACS adjustment was carried out in stages by separate blocks (the so-called “stringing”) as the field work was completed. • In total, there are 14,145 points of the old network on the territory of the Republic of Uzbekistan.

New satellite geodetic network • On the territory of the Republic of Uzbekistan in 2005-2007. points were built and measurements were made of the high-precision satellite geodetic network SGS - 0. The network consists of 20 points, including the initial point Kitab. The Kitab point is part of the World Wide Web and its ephemeris data are published daily on the Internet. • SGS - 0 points are evenly located on the territory of the Republic of Uzbekistan and provide as initial determination of satellite points of concentration for the production of topographic and geodetic and cadastral works.

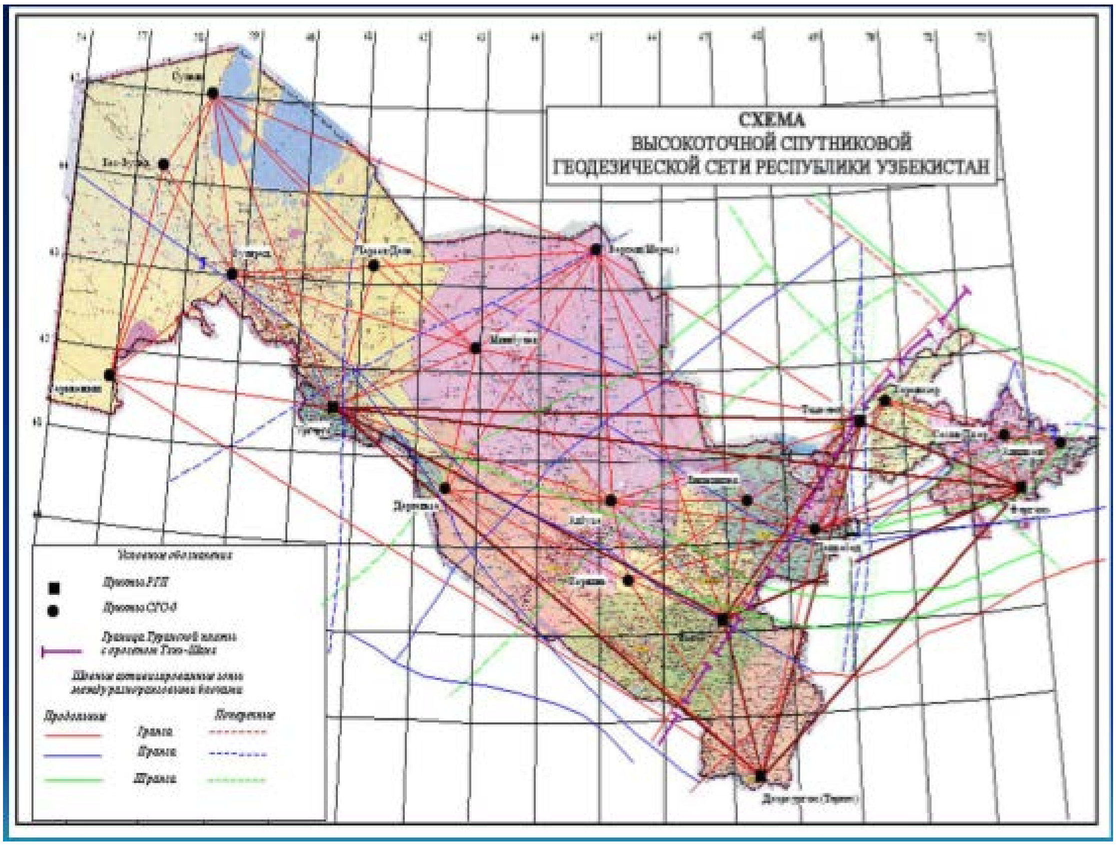


Fig.56. Scheme of high-precision satellite geodetic network of the Republic of Uzbekistan

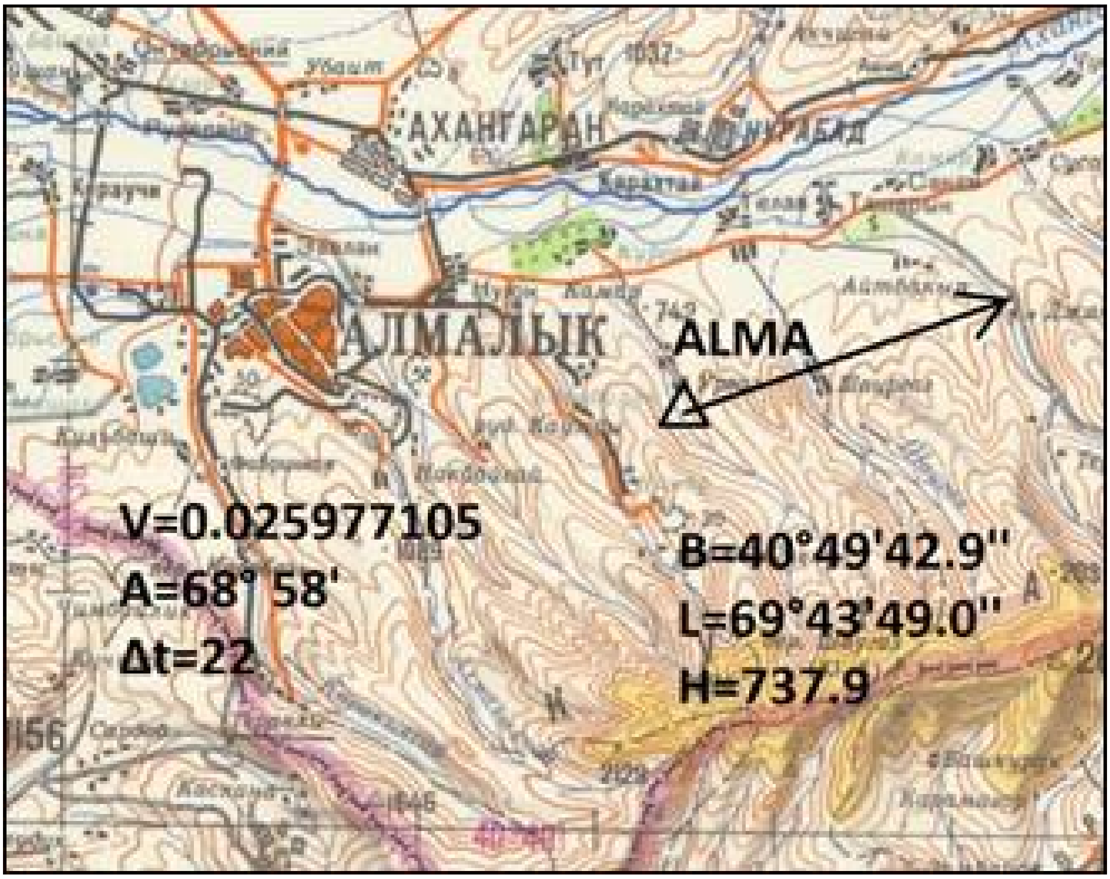
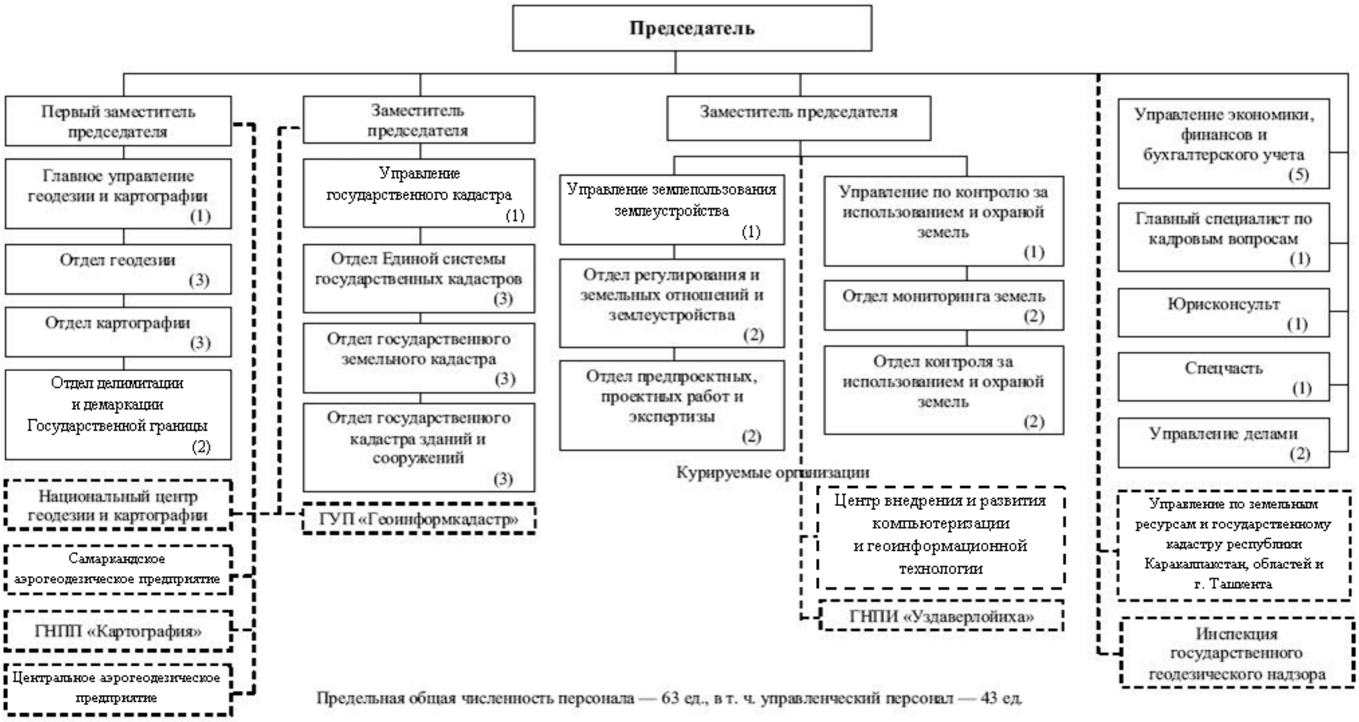


Fig.57. Designation on topographic maps of GGS points

The structure of the cartographic service of the Uzbekistan:



Law on Geodesy and Cartography

Article 1. Purpose of this Law

This Law establishes the legal framework for activities in the field of geodesy and cartography in the Republic of Uzbekistan and is aimed at creating conditions for meeting the needs of the state, legal entities and individuals in geodetic and cartographic products.

Article 2. Legislation of the Republic of Uzbekistan on geodesy and cartography

Regulation of relations in the field of geodetic and cartographic activities is carried out by this Law and other acts of the legislation of the Republic of Uzbekistan.

If an international treaty of the Republic of Uzbekistan establishes other rules than those provided for in the legislation on geodesy and cartography, the rules of the international treaty are applied.

Article 3. Objects and subjects of legal relations in the field of geodetic and cartographic activities

The objects of geodetic and cartographic activities are the territory of the Republic of Uzbekistan, outer space, including natural celestial bodies.

The subjects of geodetic and cartographic activities include state authorities and administrations that, in accordance with their competence, regulate geodetic and cartographic activities, legal entities and individuals who are customers or executors of geodetic and cartographic works or holders of geodetic and cartographic materials (data), as well as implementing geodetic and cartographic products.

Article 4. Competence of the Government of the Republic of Uzbekistan in the field of geodetic and cartographic activities

Government of the Republic of Uzbekistan:

coordinates the geodetic and cartographic activities of all its subjects on the territory of the Republic of Uzbekistan;

organizes the execution of geodetic and cartographic works for national purposes, as well as works of special (sectoral) purposes on orders from state authorities and administration;

establishes unified state systems of coordinates, heights, gravimetric measurements, a scale series of state topographic maps and plans;

determines the state body that manages geodetic and cartographic activities;

performs other functions within its competence.

Article 5. Main tasks of the state geodetic supervision

The main tasks of the state geodetic supervision are:

supervision of compliance by all subjects of geodetic and cartographic activities with the requirements of regulatory and technical documents;

supervision of the correct display of the state border and territory of the Republic of Uzbekistan on cartographic plans and other documents;

registration of geodetic and cartographic works;

accounting of geodetic points;

ensuring the functioning of the state cartographic and geodetic fund;

maintaining an on-duty reference card of the Republic of Uzbekistan.

Officials of the state geodetic supervision for the implementation of the tasks provided for in part one of this article have the right to:

demand the elimination of identified violations of the procedure for organizing and performing geodetic and cartographic work, as well as the concentration, accounting, storage, use and sale of materials (data) obtained as a result of these works and aerospace surveys;

access to enterprises, institutions and organizations to familiarize themselves with all the necessary documents on geodetic and cartographic activities;

receive from enterprises, institutions and organizations the information necessary for the implementation of the functions provided for by this Law.

The regulation on state geodetic supervision in the Republic of Uzbekistan is approved by the Government of the Republic of Uzbekistan.

Article 6. Geodetic and cartographic work for state purposes

State geodetic and cartographic works include:

determination of the parameters of the figure of the Earth and the external gravitational field;

creation, updating and publication of state topographic maps and plans in graphic, digital, photographic and other forms;

creation, development and maintenance of state geodetic and leveling networks;

remote sensing and geodynamic studies of the Earth;

formation and maintenance of the cartographic and geodetic fund of the Republic of Uzbekistan;

creation and maintenance of geographic information systems;

compilation and publication of general geographical, political-administrative, scientific reference and other thematic maps and atlases for intersectoral purposes, educational cartographic manuals;

geodetic, topographic, cartographic and hydrographic support for delimitation, demarcation and verification of the passage of the line of the state border of the Republic of Uzbekistan;

metrological support of geodetic, topographic and cartographic works;

standardization, accounting and streamlining the use of geographical names;

organization of serial production of geodetic and cartographic equipment.

Article 7. Geodetic and cartographic work for special purposes

Geodetic and cartographic works of special (industry) purpose include:

creation and updating of topographic plans intended for drawing up master plans for cities, towns, construction sites of various facilities, underground networks and structures, linking buildings and structures to construction sites, as well as for performing other special works;

creation and maintenance of geographic information systems for special purposes;

publication of thematic maps and atlases for special purposes;

carrying out geodetic, topographic, aerial survey and other special works during engineering surveys, construction and operation of various structures, land surveying, cadastral maintenance and other surveys.

Article 8

Normative-technical acts in the field of geodetic and cartographic activities establish the procedure for organizing geodetic and cartographic work, technical requirements for them, norms and rules for their implementation and are approved by the Government of the Republic of Uzbekistan or an authorized body.

Regulatory and technical acts in the field of geodetic and cartographic activities are mandatory for all subjects of geodetic and cartographic activities.

Article 9. Metrological support of geodetic and cartographic activities

Ensuring the unity of geodetic measurements, carrying out activities for testing geodetic measuring instruments, participating in the work on standardizing these tools and the work carried out for the mandatory certification of geodetic, topographic and cartographic products, carrying out metrological supervision in the field of geodetic and cartographic activities are assigned to the body that manages geodetic and cartographic activities.

Article 10. Financing of geodetic and cartographic activities

Geodetic and cartographic activities of the state purpose are financed from the state budget.

Geodetic and cartographic activities for special purposes are financed at the expense of the customer.

Geodetic and cartographic products, including topographic, hydrographic, aerospace, geodetic and gravimetric materials (data) obtained as a result of activities carried out at the expense of the state budget, are state property.

Article 11. Cartographic and geodetic fund of the Republic of Uzbekistan

The cartographic and geodetic fund of the Republic of Uzbekistan includes state and departmental cartographic and geodetic funds.

The State Cartographic and Geodetic Fund compiles materials (data) of national, intersectoral significance, and is under the jurisdiction of the body that manages geodetic and cartographic activities.

Departmental cartographic and geodetic funds constitute materials (data) that have a special (industry) purpose and are under the jurisdiction of the relevant authorities.

Regulations on state and departmental cartographic and geodetic funds and lists of materials (data) contained in them are approved in the manner established by the Government of the Republic of Uzbekistan.

Materials (data) of the state cartographic and geodetic fund of the Republic of Uzbekistan, referred in the prescribed manner to the composition of the Archival Fund of the Republic of Uzbekistan, are stored in accordance with the law.

Transfer to third parties of materials (data) of the state cartographic and geodetic fund of the Republic of Uzbekistan and their copying is not allowed without the permission of the authorities in charge of these materials (data).

Legal entities and individuals carrying out geodetic and cartographic activities on the territory of the Republic of Uzbekistan are obliged to transfer one copy of the geodetic and cartographic materials (data) created by them to the relevant cartographic and geodetic funds free of charge.

State geodetic supervision over the transfer of materials (data) by legal entities and individuals to the relevant cartographic and geodetic funds, storage and use of these materials (data), maintenance of the state register of departmental cartographic and geodetic funds is carried out by the body that manages geodetic and cartographic activities.

Users of the materials (data) of the Cartographic and Geodetic Fund of the Republic of Uzbekistan are obliged to ensure the safety of the received materials (data), return them within the established time limits and not disclose the information contained in them, constituting state secrets.

Payment for the use of materials (data) of the state cartographic and geodetic fund of the Republic of Uzbekistan is made in the manner prescribed by law.

Article 12. Copyright for geodetic and cartographic products

Copyright for geodetic and cartographic products, including topographic, hydrographic and aerospace survey materials, geodetic and gravimetric data obtained as a result of geodetic and cartographic activities, are regulated by law.

Article 13. Licensing of geodetic and cartographic activities

Geodetic and cartographic activities are subject to licensing in the manner prescribed by law.

The implementation of geodetic and cartographic activities by legal entities and individuals without a license or in violation of the conditions specified in the license entails liability in accordance with the law.

Actions of state bodies and their officials on licensing geodetic and cartographic activities may be appealed to the court.

Article 14. Protection of points of state geodetic networks

Ground signs and centers of astronomical and geodetic, geodetic, leveling and gravimetric points, including geodetic points placed on light beacons, navigation signs and other engineering structures, are state property and are under state protection.

Owners of land, owners and users of land plots on which geodetic points are located are obliged to notify the body exercising state geodetic supervision of all cases of damage or destruction of geodetic points, as well as to provide an opportunity to drive (approach) to geodetic points during geodetic and cartographic work. .

Allotment of land plots for placing geodetic points on them is carried out in accordance with the law.

The regulation on the protection of geodetic points is approved by the Government of the Republic of Uzbekistan.

Article 15. Compensation for damage caused as a result of geodetic and cartographic activities

Damage caused to a person or property of a legal or natural person as a result of geodetic and cartographic activities is subject to compensation in the manner prescribed by law.

Article 16. Responsibility for violation of the law about geodetic and cartographic activities

Violation of the legislation on geodetic and cartographic activities entails liability in accordance with the legislation.

§ 41. Types of geodetic networks. State geodetic network (GGS). Geodetic Condensation Network (GCN). Survey Geodetic Network (SGS)

From the point of view of geometry, any **geodetic network** is a group of points fixed on the ground, for which plan coordinates (X and Y or B and L) are defined in the accepted two-dimensional coordinate system and H marks in the accepted height system or three coordinates X, Y and Z in accepted three-dimensional system of spatial coordinates.

The geodetic network of Russia was created over many decades; during this time, not only the classification of networks has changed, but also the requirements for the accuracy of measurements in them.

Geodetic networks by purpose and accuracy of construction are divided into three large groups

state geodetic networks (GGS);
geodetic thickening networks (GSN);
geodetic survey networks.

The urgent task of the current period is to create a unified classification of all existing and prospective geodetic networks that would meet international standards.

The State Geodetic Network (GGS) is the main geodetic basis for topographic surveys of all scales and must meet the requirements of the national economy and defense of the country when solving relevant scientific and engineering problems. The planned network is created by the methods of triangulation, polygonometry, trilateration and their combinations; a high-rise network is created by building leveling traverses and geometric leveling networks. The state geodetic network is subdivided into networks of the 1st, 2nd, 3rd and 4th classes, which differ in the accuracy of measuring angles, distances and elevations, the length of the sides of the network and the order of successive development.

The state geodetic network of the 1st class, also called **the astronomical geodetic network (AGS)** , is built in the form of polygons with a perimeter of about 800 ... 1000 km, formed by triangulation or polygonometric links no more than 200 km long and located, if possible, along meridians and parallels.

The state geodetic network of the 2nd class is built in the form of triangulation networks, completely covering the polygons formed by triangulation or polygonometry links with triangles.

Requirements for the accuracy of measuring horizontal angles and distances in triangulation are given in Table 17, in polygonometry - in Table 18.

Table 17. Accuracy of measuring horizontal angles and distances in triangulation.

Network class	Wed sq. angle measurement error, arc. min	Relative error of basis sides	Triangle side length, km
one	0.7	1:400,000	>20
2	1.0	1:300,000	7…20
3	1.5	1:200,000	5…8
four	2.0	1:200,000	2…5

Table18. Accuracy of measuring horizontal angles and distances in polygonometry.

Network class	Wed sq. angle measurement error, arc. min	Relative travel side error	Travel side length, km
one	0.4	1:300,000	>20…25
2	1.0	1:250,000	7…20
3	1.5	1:200,000	>3
four	2.0	1:150,000	>2

In addition, the conditions for the number of sides in the course, the length of the perimeter of the polygons, and some others must be met.

Root-mean-square errors of measurement of excesses per 1 km of travel in leveling passages and networks of classes I, II, III, IY are equal to 0.8; 2.0; 5 and 10 mm, respectively; marginal errors per 1 km of travel are taken equal to 3; 5; 10 and 20 mm respectively.

For topographic surveys, the Instructions of 1966 established the following norms for the density of GGS points:

for surveys on a scale of 1:25,000 and 1:10,000 - one point per 50 ... 60 km ^{2};

for surveys on a scale of 1:5,000 - one point per 20 ... 30 km ^{2};

for surveys on a scale of 1:2,000 and larger - one point per 5 ... 15 km ^{2}.

In hard-to-reach areas, the density of GHS points can be reduced, but not more than 1.5 times.

On the territory of cities with at least 100,000 inhabitants or occupying an area within the city limits of at least 50 km ^{2}, the density of GHS points should be increased to one point per 5 ... 15 km ^{2}.

Condensation Geodetic Networks (GCN) are a planned high-altitude substantiation of topographic surveys at scales from 1:5,000 to 1:500, and also serve as the basis for various engineering and geodetic works. They are created by triangulation and polygonometry methods. According to the accuracy of measuring angles and distances, GSS polygonometry is of the 4th class, 1st and 2nd categories (Table 3).

Table 19. Accuracy of measurement of angles and distances of polygonometry of the 4th class, 1st, 2nd category.

Network rank and class	Wed sq. angle measurement error, arc. min	Relative distance measurement error
4th grade	3.0	1:25 000
1st category	5.0	1:10 000
2nd category	10.0	1:5 000

It should be emphasized that measurements in the 4th class of GSS polygonometry are performed with a much lower accuracy than in the 4th class of the GHS.

The density of GSS points should be increased to one point per 1 km ^{2} in the undeveloped area and up to four points per 1 km ^{2} in the territory of settlements and industrial sites.

The state geodetic network of the 4th class can be considered a transitional type of networks between the GHS and the GSS.

The marks of the GSS points are determined from the leveling of the IV class or from the technical leveling.

Geodetic survey networks serve as the direct basis for topographic surveys of all scales. They are created by all possible geodetic constructions; the density of their points should ensure high quality shooting. Marks of survey network points can be obtained from technical leveling (with a relief section height h ≤ 1 m) or from trigonometric leveling (with a section height h ≥ 1 m).

On the territory of Russia, in addition to the GGS, GSS, GNS (state leveling network), there are other types of geodetic networks:

fundamental astronomical and geodetic network (FAGS);
state fundamental gravimetric network (GFGS);
Doppler geodetic network (DGS);
space geodetic network (GGS);
satellite geodetic network of the 1st class (SGS-1);
satellite differential geodetic network (SDGS).

The creation of geodetic networks of any class and category is carried out according to previously developed and approved projects. The project should draw up a network diagram (the layout of network points and their connections), justify the types of centers and signs, determine the volume of measurements and their accuracy, select instruments for measuring angles, distances, elevations, and develop a measurement methodology.

The design of triangulation, trilateration and complex arbitrary networks is usually carried out on a computer using special programs.

§ 42. The principle of constructing a state geodetic network.

When creating a state geodetic network, three main issues inevitably arise that are of fundamental importance: the choice of a scheme for constructing a state geodetic network throughout the country; establishing the density of geodetic points, as well as the accuracy of determining the relative position of adjacent points in the network. Each of these issues must be considered jointly, and from two points of view: from the point of view of solving the main scientific problems of geodesy, as well as the tasks of mapping the country's territory. This is due to the fact that when solving these problems, different requirements are imposed on the reference geodetic network. Therefore, it is necessary to find, in a certain sense, the optimal variant of building a network that allows solving the problems of both groups at the proper scientific level and with the required accuracy.

Using the methods of space geodesy, one obtains sufficiently generalized, i.e., smoothed characteristics of the figure and the gravitational field of the entire Earth as a whole. In more detail, the figure of the Earth within the territory of one country or group of countries is studied by creating astronomical and geodetic networks, in which a complex of geodetic, astronomical and gravimetric measurements is performed. Until recently, in countries with a large territory, astronomical and geodetic networks were built in the form of triangulation series, laid along the directions of meridians and parallels and forming closed polygons. As a result of the joint mathematical processing of all types of measurements performed in the astronomical and geodetic network, the heights of the quasi-geoid and its profiles along the series of triangulation of the 1st class are obtained. At the same time, the shape of the quasi-geoid surface inside each polygon remains unexplored. To eliminate this shortcoming, it is necessary to create on the territory of the country not a polygonal, but a continuous astronomical-geodesic network with a more or less uniform distribution of points throughout the territory.

For geodetic support of topographic surveys carried out for the purpose of mapping the entire territory of the country, it is necessary to build a continuous reference geodetic network on its surface. In this case, the distances between neighboring points should be much less than in the astronomical-geodesic network, especially in large-scale mapping.

Thus, in order to solve both scientific and practical problems of geodesy and cartography, it is necessary to have on the territory of the country a continuous state geodetic network with an astronomical and geodetic network allocated in it as the main and most accurate, used to solve both practical and scientific problems of geodesy, including those related to a detailed study of the shape and gravitational field of the Earth within the territory of one or a group of countries.

In higher geodesy, a certain principle or scheme for constructing a state geodetic network has developed and well taken shape , designed to solve both scientific and engineering problems of national economic importance. The state geodetic network is created in stages, stages, observing the principle of transition from the general to the particular. First, the main, i.e., astronomical-geodesic network, is built, consisting of large geodetic constructions in the form of either closed polygons or relatively large triangles. Measurements in the astronomical-geodesic network are performed with the highest possible accuracy. Then, this network is taken as the initial one and, on its basis, a second-order geodetic network is built with more detailed geometric constructions and with a lower relative measurement accuracy, however, while maintaining the magnitude of the absolute error in determining the relative position of adjacent points, as in the first-order network. This refers to the average values of errors. Further, the second-order network is taken as the original one and, on its basis, a third-order network is created with even greater detail of geometric constructions with a lower relative measurement accuracy, but, as before, with the same absolute error in determining the relative position of adjacent points. This is done until a geodetic network with the required density of points is built.

Thus, while observing the principle of transition from general to particular, the state geodetic network is inevitably subdivided into geodetic networks of different classes 1, 2, 3 ... The number of classes is recommended to be minimized to reduce the impact of errors in the initial data on the adjusted network elements of the lowest class.

§ 43. Methods for constructing planned geodetic networks. Triagulation. Trilateration. Polygonometry.

Methods for creating a planned state network.

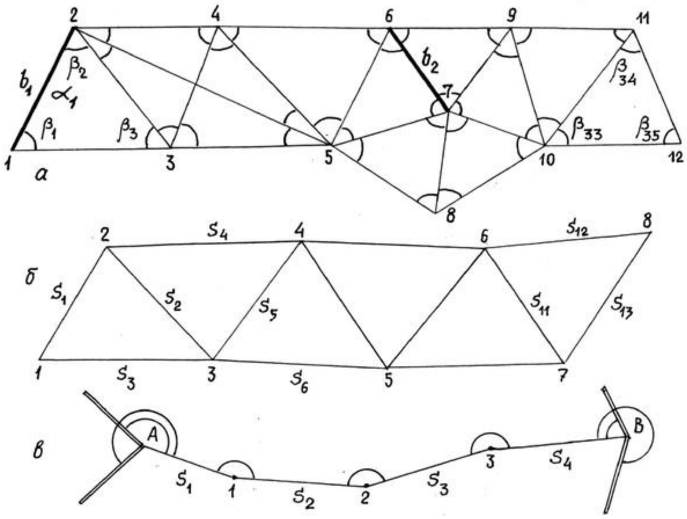


Fig.58. a) triangulation method; b) trilateration method; c) polygonometry method

1. Triangulation - a method of creating a network in the form of triangles, in which all angles and some of the sides are measured. The lengths of the rest are calculated using the sine theorem. On fig. 58 the lengths of the sides *AB* - *in* $_1$ and *CD* - *in* $_{2\ \text{are}}$ measured (bases). On the sides and directional angles, the coordinates of the vertices of the triangles are calculated.

2. Polygonometry - a method of constructing a planned geodetic network by measuring angles and distances between geodetic points (Fig. 58). The sides are measured with rangefinders.

The network is created in the form of separate passages or systems of passages.

3. Trilateration - a method of creating a geodetic network in the form of triangles (Fig. 58), the sides of which are measured by light - and radio rangefinders (more often the latter, when there is no line of sight between points). From the solution of triangles, according to the cosine theorem, the values \u200b\u200bof all angles are found.

The planned state geodetic network has been largely built by now. It is subdivided into networks of 1, 2, 3 and 4 classes, which differ from each other in measurement accuracy, distances between network points and development order (Fig. 58).

The class 1 network is used for scientific research and defines a single coordinate system for the whole country. It is a quadrangular polygons with a perimeter of 800 - 1,000 km, the sides of which are oriented mainly along the meridians and parallels. Each side (link) of a polygon 200–250 km long consists (mostly) of triangulation rows. At the junction of the links, the basic sides were measured, at the ends of which astronomical latitudes, longitudes, and azimuths were determined. Distances between points in links are 20-30 km. In some part of the territory, not a polygonal one was built (Fig. 59), but a continuous triangulation network of the 1st class with a side length in a triangle of 20 - 70 km.

All measurements are high precision. In each triangle, all angles were measured with a root-mean-square error of 0", 7. Lines 20 - 25 km long were determined with an error of 7 - 10 cm. The error in determining astronomical latitudes is 0.3", longitudes - 0.5", azimuths - 0 ,5". Relative root mean square error of the basic side 1/400 000.

Polygons of the state network of the 1st class are filled with a continuous network of triangulation (polygonometry) of the second class. The distances between the points of the sides are 7 - 20 km. In each polygon of the 1st class, 4 - 5 basic sides of the triangulation of the 2nd class are measured, evenly spaced inside the polygon. Astronomical latitudes and longitudes were measured at the ends of each base side, and geodetic azimuths were calculated. Relative rms error of measuring angles in triangulation of the 2nd class is 1.0"; relative rms error of the base side is 1/300,000. The purpose of triangulation of the second class is to be a support for geodetic networks of lower (3 and 4) classes.

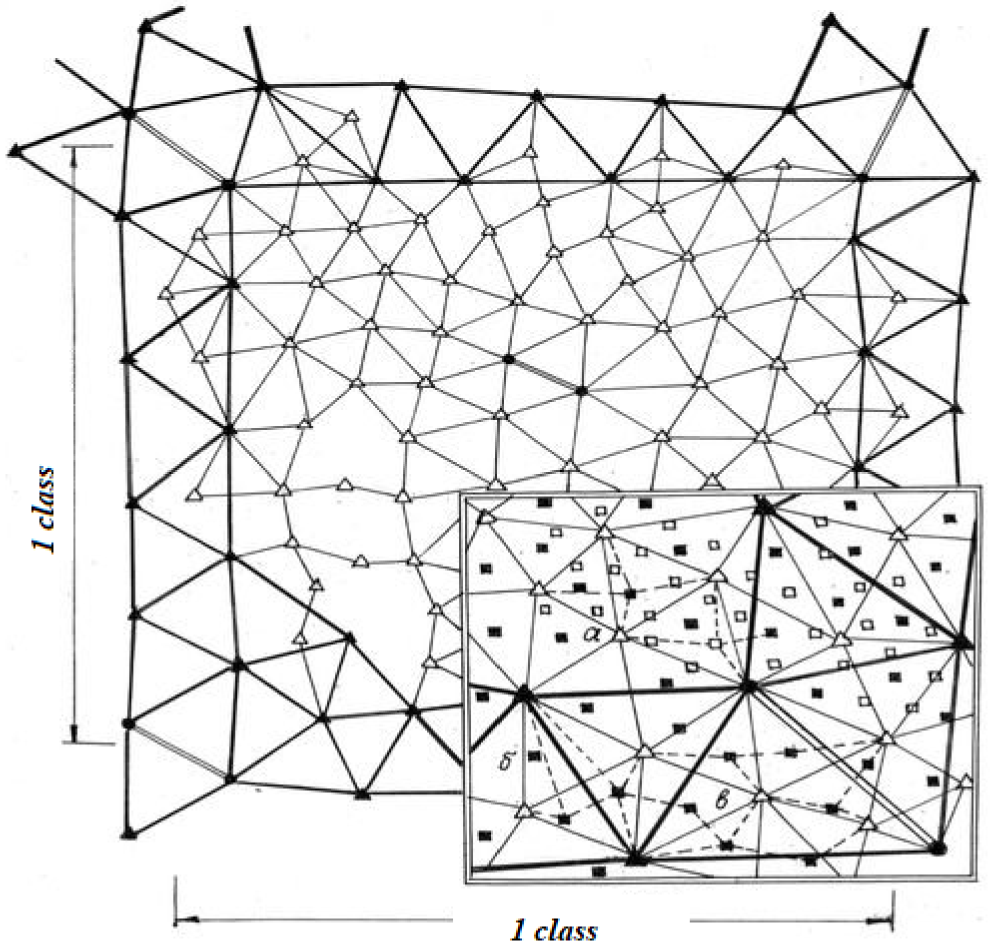


Fig.59. Scheme for constructing the HGS: Laplace points; items of the 1st class; class 2 items; class 3 items; class 4 items;

When constructing networks of classes 3 and 4, triangulation and polygonometry methods were also used. Distances between points of 3 and 4 classes respectively 5 - 8 and 2 - 5 km; angle measurement error 1.5" and 2.0" respectively. Relative root-mean-square measurement error of basic sides is 1/200,000.

For orientation, at each point of the state network there are two reference points fixed on the ground, at a distance of 250 - 1,000 m from the triangulation point, the directions to which are measured with an error of 2.5 ".

Local objects (tower spire, bell tower) located no further than 3 km from the geodetic network point and clearly visible from the ground can also be used as reference points.

Fig.60. Points of the state geodetic network

Points of the state geodetic network are fixed on the ground with special underground signs - centers, while measures are taken to help maintain their stability. So, for example, signs are laid in rocky soils, in conditions of significant moisture - in coarse-grained soils; the signs themselves are made lighter at the top, with a massive anchor at the bottom. The lower base of the sign is set 0.5 m below the boundary of the greatest freezing of the soil, and in permafrost areas - 1 m below the boundary of the greatest thawing, etc. External signs are built above the underground ones - pyramids, signals - providing mutual visibility between points and being good landmarks on terrain. They can be wooden, metal, less often - reinforced concrete.

Table 20 **Some characteristics of triangulation**

Basic triangulation options	Grade 2	3rd grade	4th grade
m_{β}	1″	1.5″	2″ _ _ _
m_{in}/in	1/ 300,000	1/200,000	1/100,000
side length, km	8 – 20	5 – 8	2 - 5

§ 44. State leveling networks.

This network establishes a unified system of heights throughout Russia and is the basis for topographic surveys and geodetic measurements performed to solve scientific and practical problems.

It allows you to: solve the problem of creating a network of points with known heights; determine the difference in levels of internal and external seas and oceans; study the current movements of the earth's crust.

For the initial reference level surface in the USSR, the surface of the Baltic Sea was taken - the surface passing through the zero of the Kronstadt footstock.

The state leveling network is divided into networks I , II , III , and IV classes.

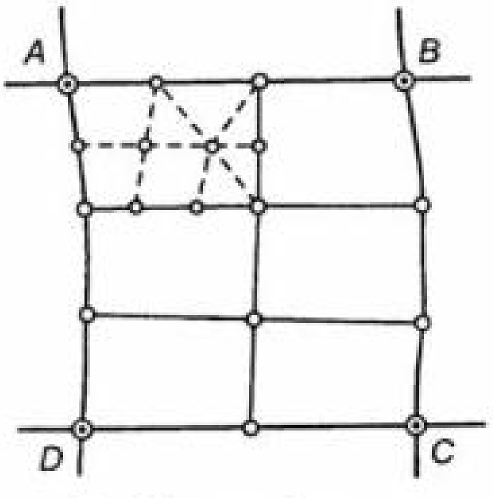


Fig. 61. Scheme of networks I , II , III and IV classes.

Leveling classification

The leveling network is created by geometric leveling in the form of separate passages or closed passages - "polygons".

The state leveling high-rise network is created according to a specially designed scheme and is of I , II , III , IV classes, then technical leveling takes place. The leveling accuracy is characterized by the root mean square error per 1 km of travel and is denoted by η. The allowable discrepancy of excesses in leveling moves is taken equal to twice the root mean square error. The table shows the characteristics of leveling accuracy of various classes.

Table 21 Characteristics of leveling classes

Leveling class	Limit perimeter of the polygon (travel length), km	Root mean square error per 1 km of double run, η , mm	Permissible discrepancy of a polygon (travel), mm
I	–	0.5	–
II	500 -	600	2.0 $5\sqrt{L}$
III	150 – 200	5.0	$10\sqrt{L}$
IV	25	10.0	$20\sqrt{L}$
Technical leveling		25.0	$50\sqrt{L}$

Here L is the length of the leveling run in km.

I leveling moves are laid according to a specially developed program that provides for:

- creation on the territory of the country of a network of starting points for the development of leveling II and other classes;
- ensuring communication of water measuring posts of the seas and oceans;
- study of geodynamic processes of vertical movements of the earth's crust.

Leveling courses of class II are laid in the form of polygons with a perimeter of 500 - 600 km. They rely on class I benchmarks.

III leveling moves divide the class II polygon into 6–9 polygons with a perimeter of 150–200 km.

Class I leveling is carried out in the directions, the geographical location of which best meets the solutions of the above-mentioned tasks. To ensure the highest accuracy, leveling is carried out along the routes of railways, highways and improved dirt roads, and in hard-to-reach areas - along paths, zaimka, along the banks of large rivers. In order to modernize and obtain data on modern movements of the earth's crust, re-leveling is carried out in 25 years for all class I lines and some class II lines .

Leveling points of all classes are fixed on the ground with signs - deep, ground (Fig. 63) or wall *leveling benchmarks* . They are laid after 5 - 7 km. In addition, points of I and II classes are fixed by especially stable fundamental benchmarks after 50 - 80 km.

The construction of the state leveling network of Russia has been basically completed by now.

Condensation Geodetic Networks **.**

These networks are created in those cases when the density of points of the state geodetic network does not provide a solution to specific research problems. They are divided into analytical networks of the 1st and 2nd category, developed by the triangulation method, polygonometric networks, developed by polygonometric methods. High-altitude thickening networks are technical leveling networks created by the geometric leveling method.

Geodetic networks of local importance.

They serve to further thicken the state planned geodetic network of 1, 2, 3 and 4 classes. Develop as needed. They are created by triangulation and polygonometry methods and come in 1 and 2 digits.

Table 22

Indicators	1 rank	2nd category
RMS angle measurement error	5"	10"
Relative error basic sides	1/50000	1/25000

Planned filming justification

It is built in the development of networks of higher orders in order to thicken the network to the density required for a given survey scale, as well as to create a rationale for engineering and geodetic work (at all stages of pipeline construction or for the development of a large field).

Creation Methods:

1. Microtriangulation.
2. Theodolite moves (analogy of polygonometry).
3. Geodetic serifs (direct, reverse, combined).

§ 45. Fixing points of geodetic networks on the ground.

The points of planned geodetic networks are fixed on the ground by installing a special center, which is laid to a depth exceeding by at least 0.5 m the depth of soil freezing, or by at least 1 m the seasonal depth of thawing soil in permafrost areas. In the upper part of the center, a brand is reinforced, on which there is a mark in the form of a hole with a diameter of 2 mm. This label also refers to the coordinates of the point. There are standard types of centers for different regions of the country and conditions for laying the center.

A signal is set above the center, the axis of the sighting cylinder 1 of which coincides along a plumb line with the brand mark.

It is very important in the construction and operation of the point to ensure the stability of the center itself and the signal. In the first case, stability is determined by the properties of the soil, changes in its moisture content, the presence of groundwater, and the possible impacts of man and nature. In the second case, both by the characteristics of the grounds of the signal base and by periodic effects of wind load on it (especially at the time of observations), heating by sunlight, exposure to humidity, etc., which causes bends, vibrations, trembling and torsion of the signal structure. Studies have established, for example, that under the influence of temperature, in some cases, the rotation of the signal in azimuth during the working day can reach several arc minutes. With measurement accuracy, for example, from 0.7 "to 5.0" - this is a very significant value.

In geodetic networks, various designs of signs are used: a simple pyramid, a pyramid with a tripod, a simple signal, a complex signal, and a tour.

Simple pyramids and pyramids with a tripod are built in cases where there is a direct line of sight to neighboring signs from the ground (from a portable tripod). If the device needs to be raised above the ground by 2-3 m, then a simple pyramid is built with a tripod 2 isolated from it. The observer moves at the table along a special flooring 3, fixed on the pillars of the pyramid. The supports of the pyramid are fixed into the ground to the anchor 4 .

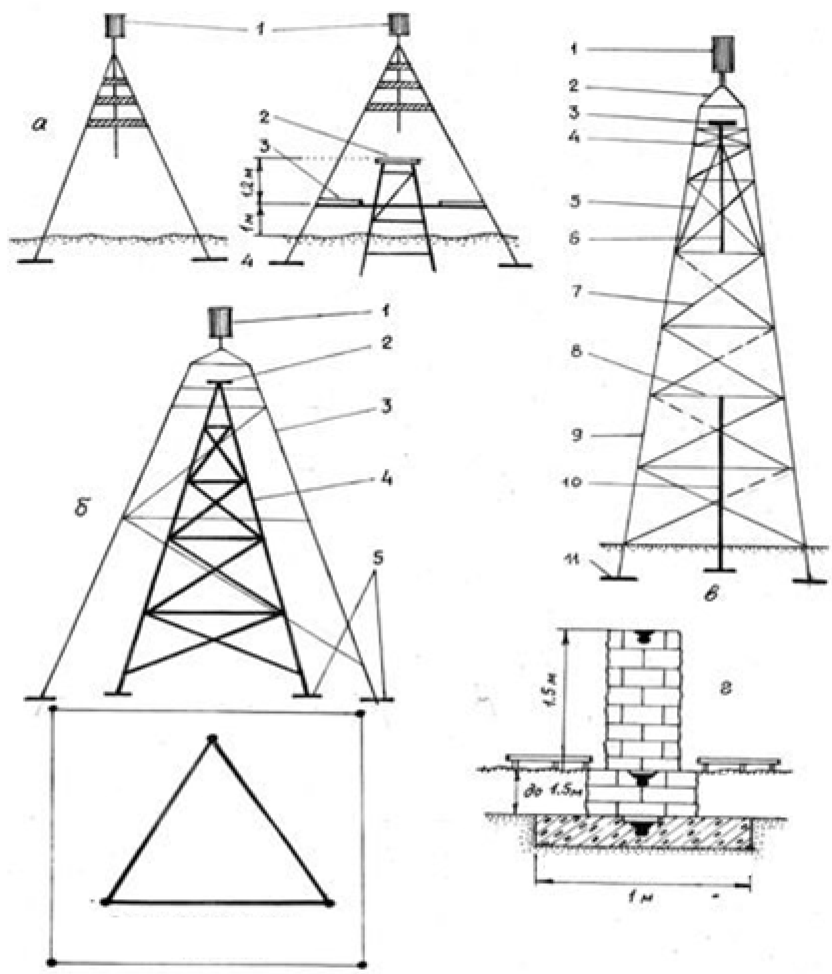


Fig.62. Signal structures: a - simple pyramids; b - simple signals; c - complex signals; g - tours.

Simple signals are used in cases where the device must be raised above the ground to a height of 4 to 10 m. A simple signal consists of two isolated structures: external 3 and internal 4, which has a platform 2 for the observer. The outer part has four supports, the inner part - three supports, fixed with anchors 5 in the ground.

Simple signals can be wood or metal. They can also be permanent and collapsible. Collapsible signals are transported from point to point in areas where there are no obstacles to the use of transport.

Complex signals have a significant height. They are built when the device should be raised to a height of 11 to 40 m. The inner pyramid 5 of the complex signal does not rest on the ground, but on the structure 9 of the outer pyramid. On the inner pyramid there is a table 3 for installing the device. The observer is located on a special platform 4.

The height of the inner pyramid is about 7 - 7.5 m. The strength of the structure is ensured by the connections formed by the crosses 7, crowns 8, fastened to the main pillars 9. The inner pyramid has its own rack 5 with a blank 6. Fragment 2 is called the roof of the sign. Element 10 is an intermediate column of the sign. The supports of the outer pyramid and the intermediate pillar of the sign are fixed in the ground on anchors 11.

Composite signals are currently made only trihedral, which facilitates their complete assembly on the ground and installation in the working position already fully assembled.

Tours are installed in places where there is rocky soil at a depth of no more than 1.5 m, and good visibility is provided in all directions necessary for measurements. A simple pyramid with a sighting cylinder is installed above the tour. Sometimes the sighting cylinder is fixed directly on the tour. During measurements on such tours, the sighting cylinder is temporarily removed .

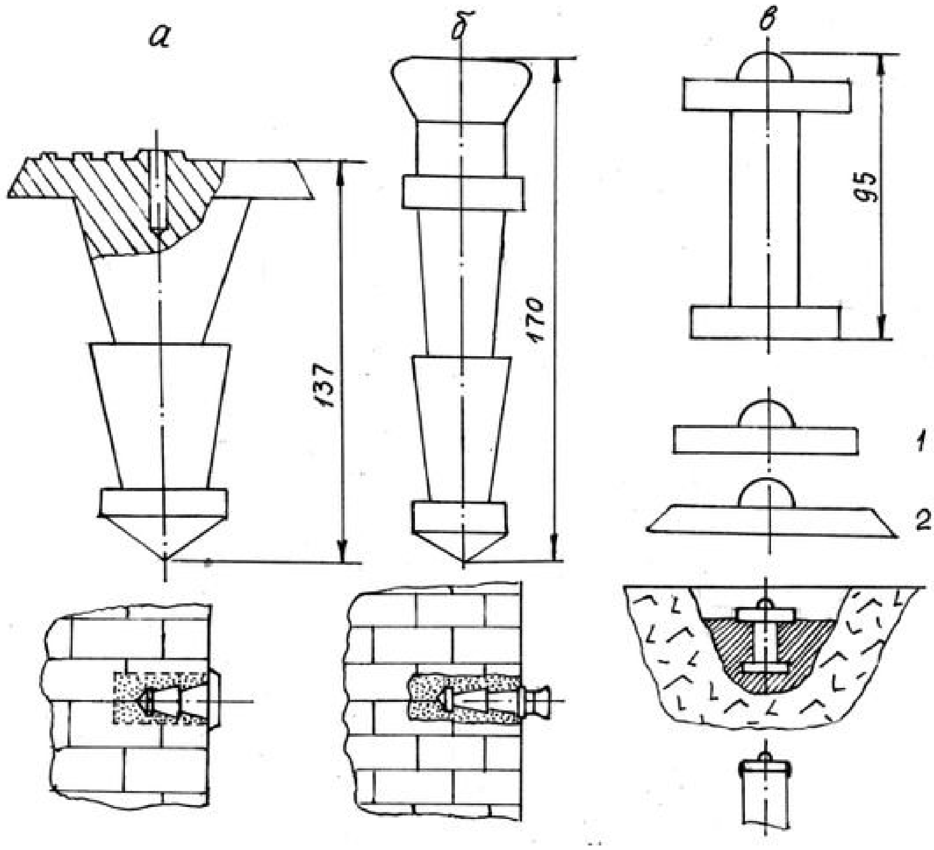


Fig.63. Types of benchmarks of the leveling network: a - wall mark; b - wall benchmark; g - brand for concrete and rock benchmarks; 1 and 2 - grades for installation on tubular benchmarks

An inscription containing the abbreviation of the organization and the number of this benchmark is placed on the disks of stamps or benchmarks.

Chapter 10
Theodolite, tacheometric surveys The concept of ground, aerial photogrammetric and satellite surveys .

§ 46. Theodolite survey. Outline of theodolite survey.

Theodolite is a horizontal (contour) survey of the terrain, as a result of which we get a plan depicting the situation of the terrain (contours and local objects) without relief.

Shooting scale: 1:5000-1:500.

Shooting is used on flat terrain in a difficult situation and in a built-up area; in settlements, on construction sites.

The points of the theodolite traverse serve as the points of the planned survey justification.

A theodolite traverse is a broken line where turning points are fixed on the ground with temporary signs (wooden pegs), between which distances and horizontal angles are measured. They are divided by accuracy and form.

By accuracy, theodolite moves are divided into moves of the 1st and 2nd categories:

• strokes of the 1st category are characterized by a relative error of at least 1/2000;

• moves of the 2nd category - with a relative error not worse than 1/1000.

According to the form of construction, the following types of moves are distinguished:

- open stroke (the beginning and end are based on the points of the backbone network - Fig. 65, a);
- closed move - a closed polygon, usually adjacent to the point of the reference network (Fig. 65.b);
- hanging passage - one of the ends of which is adjacent to the point of the support network (Fig. 65, c).

The choice of the form of the course depends on the nature of the territory of the object. The laying of hanging passages is allowed in some cases when shooting non-critical objects.

In terms of content, theodolite survey includes preparatory, field and cameral work .

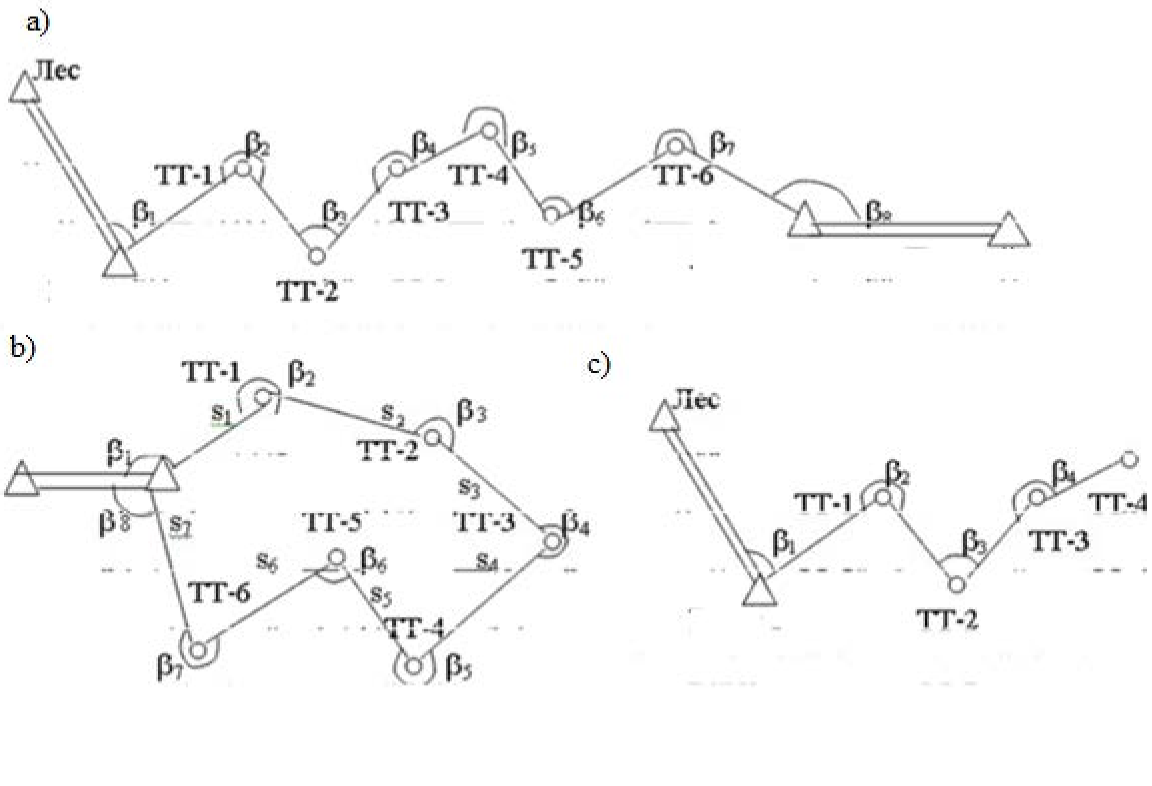


Fig.65. Types of theodolite passages: *a* - open passage; *b* - closed course; *c* - hanging move

Preparatory work includes the following steps :

- selection of the survey scale based on the required accuracy of the situation display, selection and study of available plans, maps, profiles, reference network;

- drawing up a diagram of the core network;
- drawing up a scheme of theodolite passages (development of a preliminary draft of field work);
- preparation of instruments (theodolite, tape, pegs).

Stages of field work :

- reconnaissance of the area and fixing points of theodolite passages;
- binding of theodolite moves to the points of the reference network;
- laying theodolite passages on the ground;
- shooting the situation of the area.

Cameral works include the following stages :

- calculation of the coordinates of the traverse vertices;
- building a theodolite survey plan.

The sequence of field work:

1. Reconnaissance of the area and fixing the points of theodolite traverses provides for: inspection of the area in order to get acquainted with the objects of survey; finding points of the reference network, the final choice of points of the theodolite traverse.

Requirements when choosing points of the theodolite traverse: mutual visibility between adjacent points; convenience of measuring the distance between points; side lengths 20-350 m; the slope angles of the terrain are minimal - up to 50 ppm.

Fixing the points of the move is shown in fig. 66.

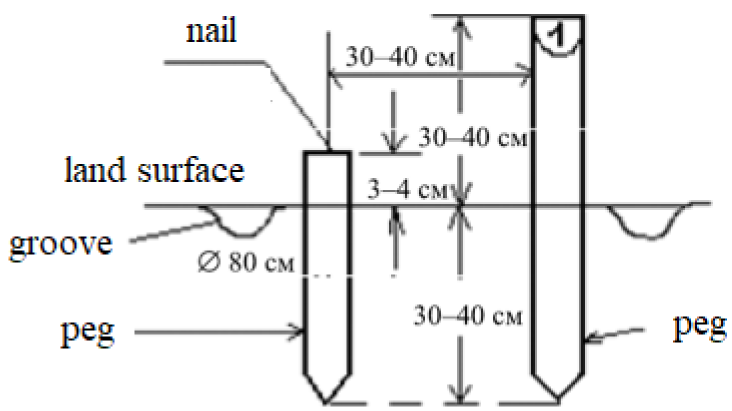


Fig.66. Fixing waypoints

2. Binding of theodolite traverses to the points of the reference network is carried out to obtain the coordinates of the traverse points in the selected coordinate system and to control measurements.

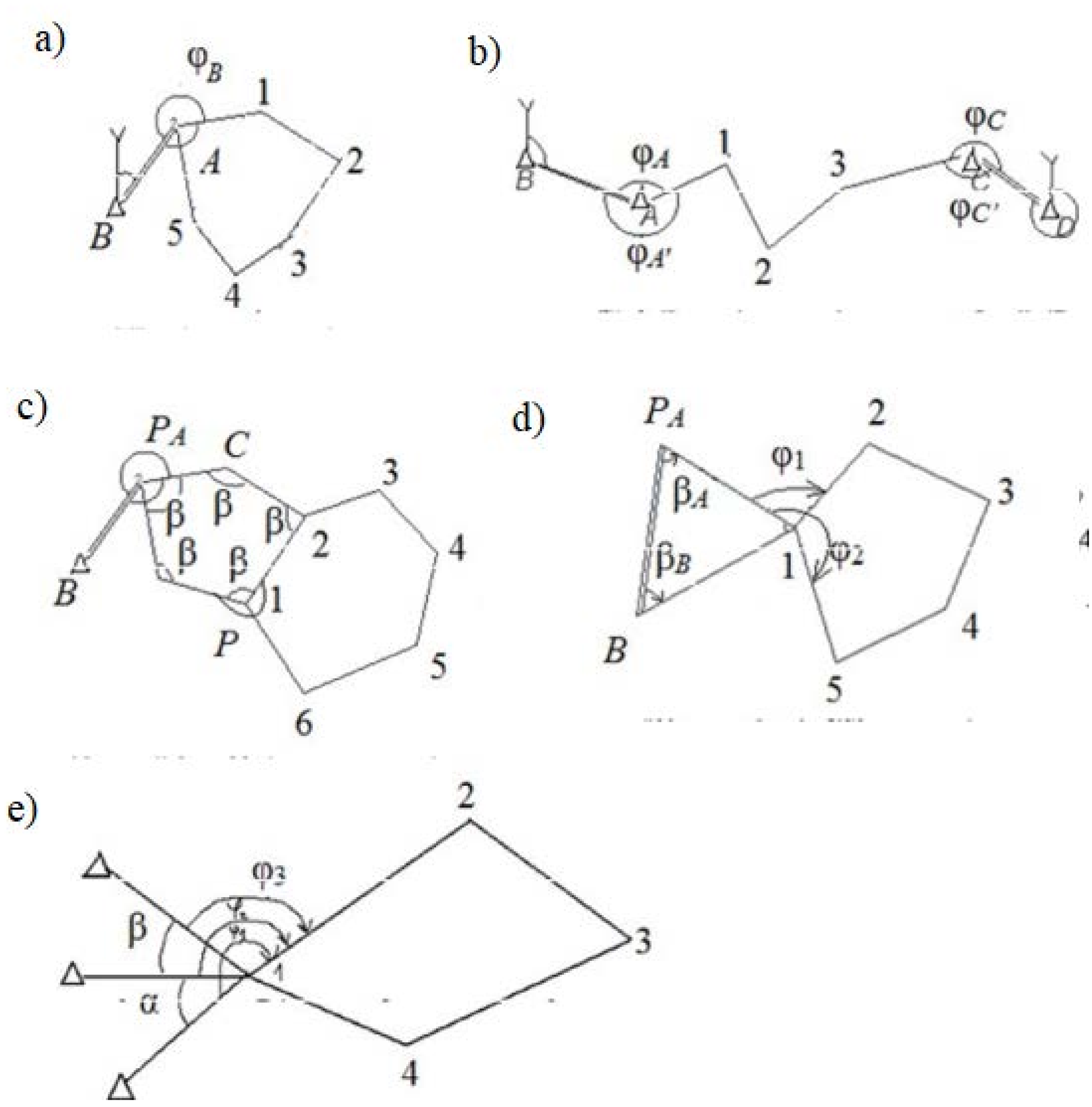
The essence of the binding is to transfer from the points of the reference network of coordinates and the directional angle to one of the points of the theodolite traverse.

The coordinates of the starting points and orientation directions *(X, Y,* os) are written out from the catalog of coordinates.

Binding options are shown in fig. 67.

3. In the course of laying a theodolite traverse on the ground, angular and linear measurements are made.

In the process of angular measurements, horizontal angles and tilt angles are measured with theodolites. Horizontal angles are measured in one full stroke.



Rice. 67. Binding of theodolite traverses to the points of the GGS: *a* - the theodolite traverse directly adjoins the point of the GGS; *b* - theodolite traverse is laid between two points of the GGS; *c* - the theodolite traverse does not adjoin the reference network. From the nearest GGS point, a special tie-in is laid; *d, e* - from the nearest points of the GGS decide a resection

Theodolite centering is performed by a plumb line or an optical plummet with an error of no more than 5 mm (the shorter the sides and the closer the angle p to 180 °, the more carefully you need to center).

When aiming at a milestone, sighting is carried out at the base of the milestone.

All entries are kept in a field journal. With linear measurements, the lengths of the lines are measured with comparated instruments (optical rangefinders) twice with control. In this case, the relative errors should not exceed the following values: for the course of the 1st category - 1:2000 (terrain with average conditions), for the course of the 2nd category - 1:1000 (terrain with unfavorable conditions). Measurement of line inclination angles at angles of inclination less than 5° ($v < 5°$) can be carried out with an eclimeter or a theodolite, over 5° - with a theodolite. Inaccessible distances are measured indirectly.

4. Shooting the situation of the terrain is to determine the position of the points of the contours and local objects relative to the tops and sides of the theodolite traverse.

The survey can be carried out simultaneously with laying the traverse or independently. The survey results are entered into a schematic drawing - an outline (Fig. 68).

The outline is a field survey document and serves to draw up a plan of the area.

Shooting methods: perpendiculars, polar coordinates, bipolar coordinates (serifs); alignments (measurements) (Fig. 68.).

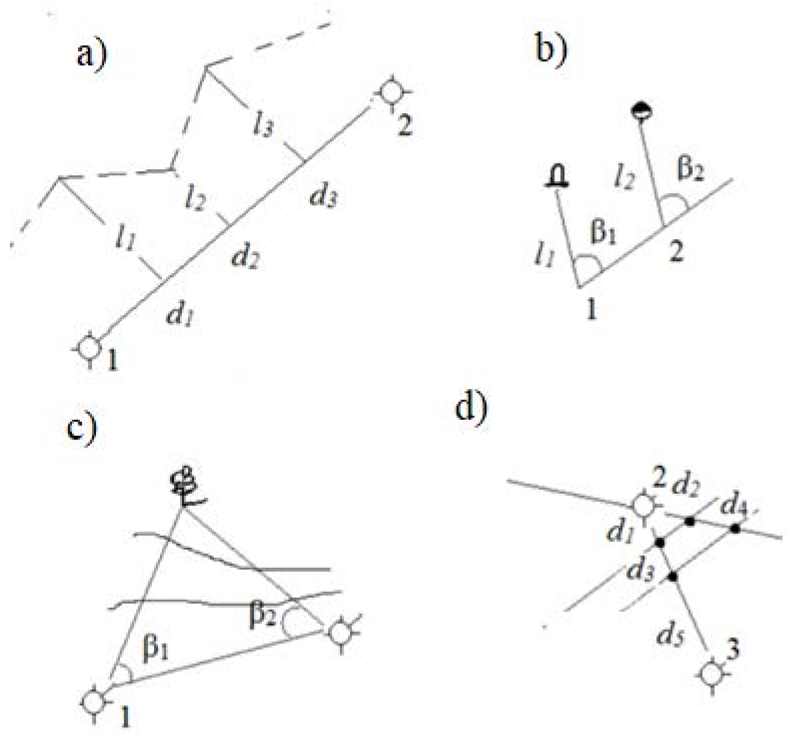


Fig. 68. Ways of shooting the situation: *a* - the method of perpendiculars; *b* - polar coordinates; *c* - bipolar coordinates (serifs); *d* - alignments (measurements)

Office work during theodolite survey includes computational and graphical processes.

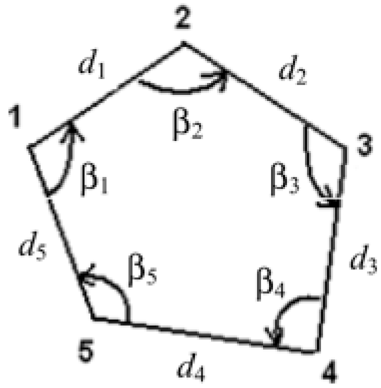
The purpose of the calculations is to obtain the coordinates of the traverse points.

The purpose of the graphical process is to obtain a plan of the area.

As a result of the inevitable errors in the measurements, there is a difference between the actual and theoretical results - a discrepancy, the distribution process of which is called equalization.

§ 47. Office processing of field measurements. Residual distribution.

Processing measurement results in closed traverse



Rice. 69. Scheme of a closed theodolite traverse

Processing of measurement results in a closed theodolite traverse (Fig. 75) is carried out in the following sequence:

- 1. Processing of angular measurements.
- 2. Determination of the sum of the measured Zp meas and the sum of the theoretical angles Khrtsor.
- 3. Calculation of the angular discrepancy / in .
- 4. Determination of corrections to the measured angles 5p.
- 5. Determination of corrected angles Zp H cn.
- 6. Control of the sum of corrected angles: Zp H cn = ^Pmcor .

$$\sum\beta_{\text{изм}} = \beta_1 + \beta_2 + \ldots + \beta_n;$$
$$\sum\beta_{\text{теор}} = 180^\circ(n-2);$$
$$f_\beta = \sum\beta_{\text{изм}} - \sum\beta_{\text{теор}};$$
$$f_{\beta_{\text{доп}}} = 1{,}5\sqrt{n};$$
$$\left|f_\beta\right| \le f_{\beta_{\text{доп}}};$$
$$\delta_\beta = -\frac{f_\beta}{n};$$

$$\sum\delta_\beta = -f_\beta;$$
$$\beta_{\text{испр}_i} = \beta_{\text{изм}_i} + \delta_\beta;$$
$$\sum\beta_{\text{испр}} = \sum\beta_{\text{теор}}.$$

7. Calculation of directional angles of the sides of the course:

$$\alpha_i = \alpha_{i-1} \pm 180^\circ - \beta^{\text{прав}}_{\text{испр}i}. \quad (115)$$

8. Calculation of horizontal distances of line lengths (Fig. 70).

9. Calculation of increments of coordinates (Fig. 71).

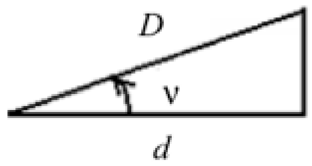


Fig. 70. Calculation of horizontal distances of line lengths

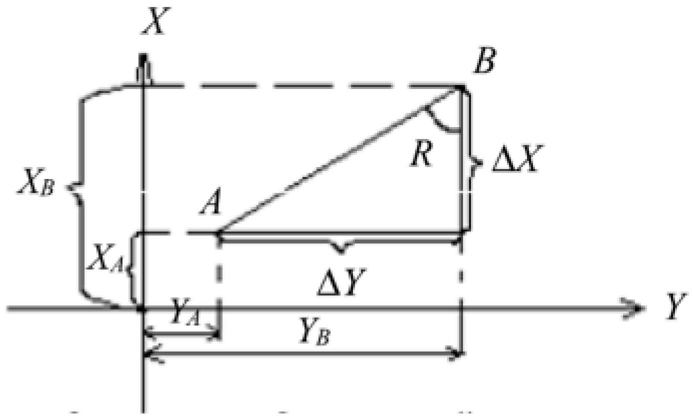


Fig. 71. Calculation of increments of coordinates

By solving the direct geodesic problem, the coordinates of the points of the course are calculated:

$$X_B = X_A + \Delta X; \quad Y_B = Y_A + \Delta Y;$$
$$\Delta X = d \cos \alpha (R); \quad \Delta Y = d \sin \alpha (R). \quad (116)$$

10. Calculation of the linear discrepancy.

The calculation of the linear discrepancy is carried out in order to compare the calculated coordinates with the control ones, determine the error of the work performed, and distribute the discrepancy. The calculation is carried out in the following sequence.

Residuals are determined in increments of coordinates:

$$\begin{array}{ll} \sum \Delta x(+) = & \sum \Delta y(+) = \\ \sum \Delta x(-) = & \sum \Delta y(-) = \\ \hline f_x = & f_y = \end{array} \quad (117)$$

The relative discrepancy of the course is calculated:

$$f_{абс} = \sqrt{f_x^{\ 2} + f_y^{\ 2}};$$

$$f_{отн} = \frac{f_{абс}}{P} = \frac{1}{P : f_{абс}} = \frac{1}{N};$$
$$f_{отн} \le f_{доп.отн} \le 1:2000 \ldots 1:1000. \quad (118)$$

The corrections in the increments of the coordinate are determined:

$$\delta x_i = -\frac{f_x}{P} \cdot d_i; \quad \delta y_i = -\frac{f_y}{P} \cdot d_i;$$
$$\sum \delta x = -f_x; \quad \sum \delta y = -f_y. \quad (119)$$

Corrected increments are calculated:

$$\Delta x_{\text{испр}i} = \Delta x_i + \delta x_i; \quad \Delta y_{\text{испр}i} = \Delta y_i + \delta y_i. \quad (120)$$

The amount of corrected increments is controlled:

$$\sum \Delta x_{\text{испр}} = 0; \quad \sum \Delta y_{\text{испр}} = 0. \quad (121)$$

The coordinates of the travel points are determined:

§ 48. Office work in the preparation of theodolite survey results

Sequence of work:

1. Construction of a coordinate grid.
2. Drawing points of the theodolite traverse on the plan.
3. Putting the situation on the plan.
4. Making a plan.

Building a coordinate grid:

dimensions of the coordinate grid for large-scale plans - 10x10 cm;
grid construction can be performed using a scale ruler, a compass, a Drobyshev ruler, or a coordinate graph.

Drawing points of the theodolite traverse on the plan is carried out in the following sequence:

determine the square in which the point is located;
using a transverse scale and a measuring compass, set aside the segments corresponding to the increments Ax, A *y*.

Drawing the situation on the plan is made from the sides and vertices of the theodolite traverse according to the survey outline. In the process of work, a scale ruler, a compass, a protractor are used.

Making a plan.

The final design of the plan is carried out in ink (pens) in compliance with the rules of topographic drawing.

The situation is indicated by conventional signs for topographic plans.

The frame design of the plan includes: the name of the document, the linear and numerical scale, the date of shooting, the artist.

Theodolite passages are laid on the ground and are used as a planned justification when tracing linear structures, performing topographic surveys, and marking work.

The outline of the theodolite survey depicts the corresponding points and sides of the survey justification, the entire situation taken from them, and, most importantly, the results of all field geodetic measurements are recorded, shown in such a way that it is clearly clear in what way and from which points and sides of the survey justification the survey of one or more other contour point of the area.

§ 49. Tacheometric survey.

A total station is a device that measures horizontal angles, distances and elevations.

The purpose of the survey is to obtain a topographic plan (that is, a plan with contours and contour lines).

The shooting rationale for tacheometric surveys are:

1. Theodolite - leveling moves.
2. High-altitude theodolite passages.
3. Tacheometric moves.

The idea of tacheometric survey.

The planned position of the points is obtained by the polar method (Fig. 72), by measuring the polar angle β along the horizontal circle of theodolite, and the distance D using a filament rangefinder. Simultaneously measure the angle of inclination ν along the vertical circle of the theodolite to determine d and h .

As reference points (stations) from which the survey is carried out, points of the state geodetic network, the concentration network, and also the survey geodetic network can be used. The latter for tacheometric survey can be made in the form of theodolite - leveling moves (the planned position of the points is determined by the method of laying theodolite moves, and the heights of the points from geometric leveling); theodolite - high-altitude passages, in which heights are determined from trigonometric leveling; and tacheometric moves, which differ from the previous ones in that here the measurements are performed by electronic total stations.

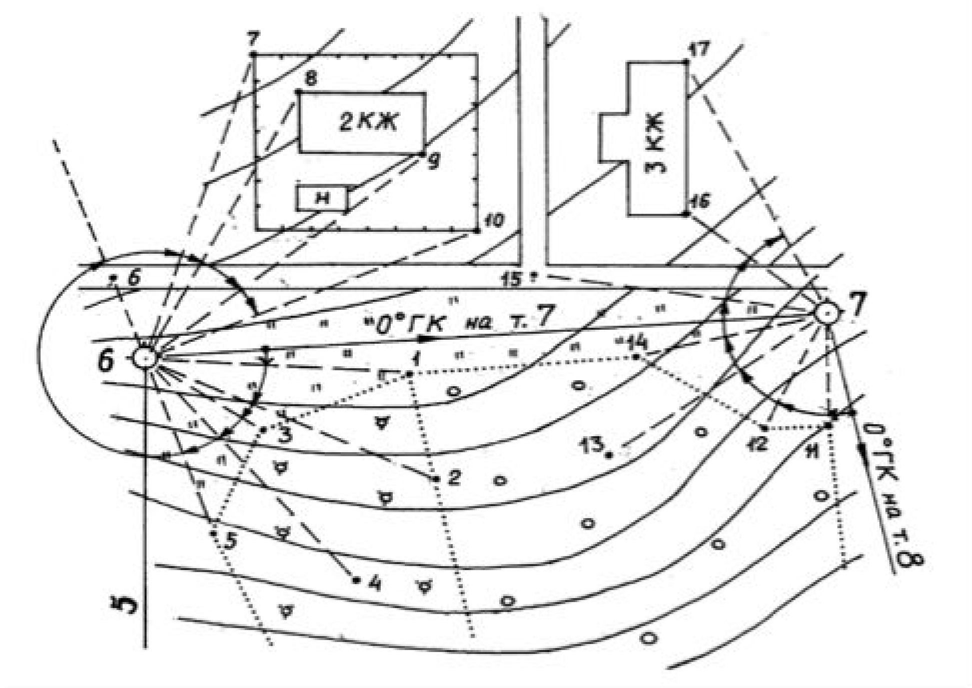


Fig.72. Scheme of tacheometric survey at stations 6 and 7.

Operating procedure

Install the theodolite at point 6 (Fig. 68), orient and fix the limb, measure the height of the device above the installation point. Using a rail mounted on a point in the terrain, using a filament rangefinder, determine the distance *D* , measure the angle *β in a horizontal circle* , and the angle *v in a vertical circle* .

The excess *h* is calculated using the trigonometric leveling formula:

$h = d \cdot tg\,\nu + a - l$.

The measurement results for each point are recorded together with the point number in the tacheometric survey log.

The points at which the rail is installed are chosen in such a way that, with a minimum number of them, the situation and relief are correctly depicted on the plan.

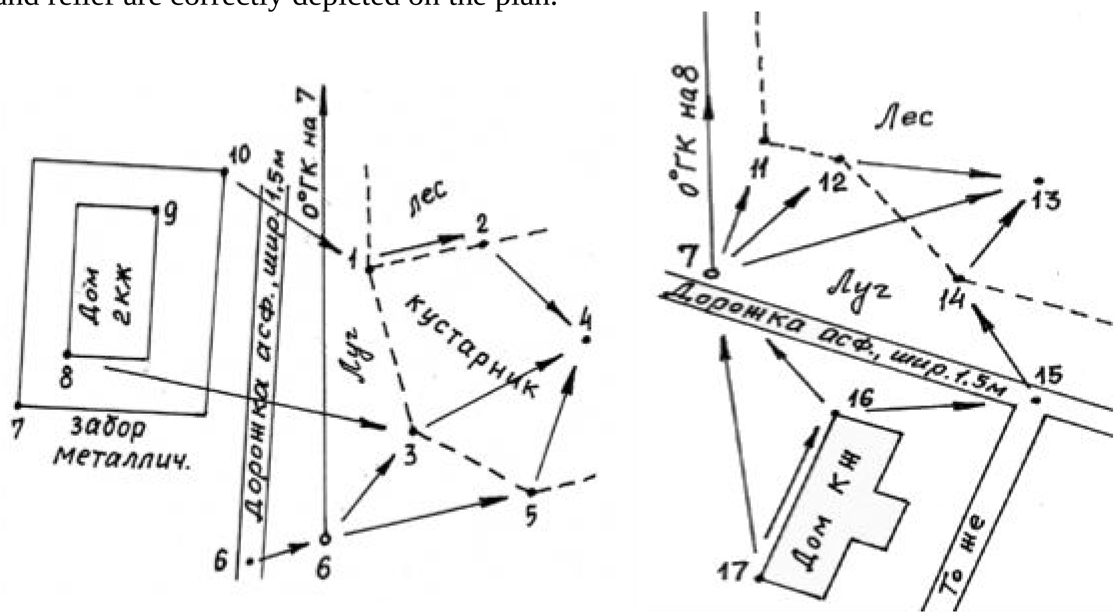


Fig.73. Outline of the tacheometric survey at stations 6 and 7.

In the course of work at the station, an outline is drawn up and maintained (Fig. 73), which shows the station, the direction of orientation of the horizontal circle, the situation and the location of the surveyed points. The relief is depicted either by conditional horizontal lines or by arrows connecting river points, between which the slope of the terrain does not change.

Upon completion of the survey at the station, the correct orientation of the horizontal circle is checked, for which they are re-sighted to the original direction. The discrepancy in readings should not be more than 2'.

In order to avoid large alterations, it is recommended to perform such control in the course of work at the station through 10 - 15 points with control fixation.

The tacheometric survey plan is drawn in the following sequence: a coordinate grid is built, then survey grid points are plotted along the coordinates. After that, with the help of a protractor and a scale ruler, picket points are plotted in a polar way.

The terrain is depicted by contour lines, the position of which on the plan is determined by the method of graphic interpolation, for example, as follows: parallel lines are drawn on a sheet of transparent paper (tracing paper) at arbitrary but equal distances from each other. These lines are signed in accordance with the accepted height of the relief section.

Having interpolated on the plan in all other directions marked on the outline, the position of points with known heights is obtained. Points with the same heights are connected by smooth lines and thus the position of contour lines is obtained on the plan.

§ 50. The concept of ground, aerial phototopographic and satellite surveys. Satellite geodesy.

Laser scanners are devices for topographic surveys, as well as for surveying various man-made objects. As a result of laser scanning, an array of spatial coordinates of the points of the object being photographed is obtained. The distance to each point is determined by a reflective range finder. In addition to the distance, each point has its own horizontal angle and tilt angle, and the intensity of the reflected beam is also recorded. The data obtained make it possible to build a spatial model and image of the object being filmed in a given coordinate system with an accuracy of determining the coordinates of any point of the model of the order of ±3 mm. However, to make it easier to work with the resulting object model, some scanners have digital cameras that allow you to get an image in real colors. Scanners for aerial and ground surveys have been developed.

Currently, two types of terrestrial laser scanners can be distinguished: specially designed scanners in which scanning is carried out by moving optical elements and scanners based on a non-reflective electronic total station with servo drives. The detail of shooting in both types of devices is set by the scanning step.

Table 23 lists the main characteristics of some terrestrial laser scanners.

Table 23

System name (manufacturer)	Distance to the scanner object, m	Accuracy mm / dist. m	Field angle vision in horizon. And vert. plane	Scan time, min	Operating temperatures	Safety class

Callidus 1.1	0.15 - 150	5/32		4 - 9	0 - +40°C	class1
(callidus)						
Cyrax 2500	1.5 - 100	4/50		ten	0 - +40°C	class2
(Cyra Technologies)			360°x180°			
ILRIS-30	2 - 350	10/100		eight	-20 - +50°C	class1
(optech)	at 4%		40° x 40°			
	reflected .	0.5/5				
SOISIC			40° x 40°		+5 - +40°C	Class 3 A
(MENSI)	2-800					
	at 20%	25/200	46° x 320°			
RIEGL	reflected.			fifteen	0 - +40°C	class1
LMS-2210						
	0.8 - 40	6/200	330 ° x 80 °			
RIEGL				0.5 _	0 - +40°C	Class 1
LMS-2360	2 - 350					
		20/1000	3 6 0 ° x 90°			
RIEGL	2 - 200			0.5	-20 - +50°C	Class 1
LMS-2420						
	2 - 1000	8/20	3 6 0 ° x 80°			
RIEGL				up to 2	+5 - +40°C	Class 1
LPM-25HA	1 - 40					
			18 0 ° x 150 °			
Laser Ase (R)				2		Class 1
Scanner	up to 700 m	50/700	360°x			
Leica						
				250 points/sec		

Topographic survey of water areas.

On April 30, 1982, the UN Convention on the Law of the Sea was adopted. This is a single comprehensive international legal document regulating the main issues of activity in the waters of the World Ocean, on the seabed and in its subsoil.

Going beyond the framework of the convention, as an international legal basis, is unacceptable, illegal, and in some situations criminal, as it is fraught with international conflicts. The above applies to all types of scientific research at sea, including topographic and geodetic research. Carrying out topographic and geodetic research is necessary to support work related to the exploration and development of ocean resources, work on the delimitation of maritime spaces, demarcation of maritime boundary lines, etc.

The main provisions of the convention regulating the legal status and purpose of topographic work in the waters of the World Ocean are as follows.

The normal baseline is the low tide line along the shore as shown on large scale charts officially recognized by the coastal State.

A straight baseline is a straight line joining the points established by the coastal state on the extremities of its coastline protruding into the sea, including chains of islands located in its immediate vicinity (at a distance not exceeding the width of the territorial sea) and highlands drying up at low tide.

Baselines (BL) are used to measure the breadth of the territorial sea, the exclusive economic zone and the continental shelf of the coastal State, and straight BL are used for this purpose when the coastline is deeply indented and tortuous.

Territorial sea (TM) - a sea belt outside the land territory and internal waters (bays, gulfs, estuaries, etc.) of a coastal state. On the TM, as well as on the airspace above it, its bottom and subsoil, the coastal state exercises sovereignty. The outer boundary of the TM is a line, each point of which is located from the nearest point of the FI at a distance equal to the width of the TM. The width of the TM is set within limits not exceeding 12 nautical miles.

An exclusive economic zone (EEZ) is an area of the high seas located outside the TM and adjacent to it. In the EEZ, the coastal state exercises: sovereign rights for the purpose of exploration, exploitation and conservation of natural resources in the waters covering the seabed, on the seabed and in its subsoil; jurisdiction over the creation and use of artificial islands, installations and structures, marine scientific research, environmental protection. The width of the EEZ should not exceed 200 nautical miles measured from the IL.

The baselines, as well as the outer limits of the HM and EEZ, are indicated on maps issued by the coastal state at a scale that is acceptable for pinpointing their position. Maps can be replaced by a list of geographical coordinates of points indicating the original geodetic data. Copies of maps or lists shall be deposited by the coastal State with the Secretary-General of the United Nations.

The continental shelf (CS) of a coastal state includes the seabed and subsoil of submarine areas extending beyond its TM to the outer boundary of the continental margin or to a distance of 200 nautical miles from the IL, when the outer boundary of the continental margin (continental margin) does not extend to such a distance.

The boundaries of the CS beyond 200 nautical miles from the IL are established on the basis of the recommendation of the Commission on the Limits of the Continental Shelf. The Commission develops recommendations based on data on the borders of the KSh, which will be provided to it by the coastal state within 10 years from the entry into force of the convention. Maps and geodetic information permanently describing the outer boundary of the KSh shall be deposited by the coastal state with the UN Secretary General.

The coastal state exercises sovereign rights over KSH for the purpose of exploration and exploitation of its natural resources. The noted rights do not affect the status of the overlying waters and the airspace above them. The exercise of the rights of the coastal state in relation to the KS should not infringe on the navigation, laying of cables and pipelines by other states. In addition, beyond 200 nautical miles from the IL, it may not, at its own discretion, refuse consent to other states for their scientific research.

The High Seas (OM) is a maritime area that is not included in either the HM or the EEZ. The basis of the legal regime of OM is the principle of freedom of the high seas. This means that it is open to the general equal and free use of all states.

Area - the bottom of the seas and oceans and their subsoil outside national jurisdiction. The area is open for use exclusively for peaceful purposes.

Activities in the Area are all activities for the exploration and exploitation of the resources of the Area. Activities in the area are carried out for the benefit of all mankind.

Resources means all solid, liquid or gaseous mineral resources, including polymetallic nodules in the Area. Resources extracted from the Area are considered to be minerals. The Area and its resources are the common heritage of mankind. All rights to the resources of the Area belong to all mankind, on whose behalf the "Authority" acts.

"Authority" means the International Seabed Authority that organizes and controls activities in the Area. The Authority regulates the development of the resources of the Area for the development of the world economy and the balanced growth of international trade.

The task of surveying includes obtaining quantitative and qualitative data in the required volume and with a given accuracy for constructing a topographic map or other cartographic model of the surveyed water area.

Quantitative data convey the relief of the seabed, the planned and altitudinal position of engineering structures and communications, the position of the boundaries of sea spaces and zones, the distribution of bottom sediments (soils), vegetation, and hydrophysical parameters. In quantitative terms, the lithological features of soils, the landscape and ecological state of the area, the distribution of benthic organisms (phyto- and zoobenthos), types of vegetation, etc. are characterized.

The features of the survey are due to the non-stationarity of water masses throughout their thickness, the peculiarity of the electrophysical properties of water (its opacity for most of the spectrum of electromagnetic waves), and the impossibility of a visual review of the objects being filmed.

Bottom relief is the main element of the situation depicted on topographic maps of water areas. Bottom relief survey methods are sounding, areal hydrological survey and remote sensing.

Sounding is a method of surveying the bottom relief by directly and indirectly measuring depths on tacks that cover the area being surveyed in a regular grid. The distance between survey lines (line-to-line distance) is one of the main sounding parameters, often combined with the concept of sounding detail. *The detail of the measurement* is established taking into account the general nature and dissection of the relief. With a decrease in the distance between lines (up to some optimal value), the detail of the survey increases.

As the features of the relief of the bottom surface are revealed, in the process of sounding, the tacks are thickened in those places where this is necessary.

Survey work is carried out on a scale of 1: 2,000 - 1: 50,000. For shelf areas with depths over 200 m, it is allowed to perform survey on a scale of 1: 100,000.

The detail of the survey should ensure compliance with the accepted requirements for the reliability and accuracy of the image of the relief by contour lines on the scale of the map being created. On the initially created maps, the average errors in the position of horizontals, relative to the height base, should not exceed 2/3 of the bottom topography section with slope angles up to 6 ° and the entire section height - in areas with slope angles from 6 to 20 °.

The detail of the measurement is set depending on the depth and vertical dissection of the relief in accordance with the data in Table. 24

Recommended sampling frequency

Table 24

Depth, m	Relief category		
	Undivided and weak dissected plains from average values relative depth of cuts 3 - 10 m.	dissected plains with relative depth of cuts 10 - 50 m.	dissected plains with relative depth of cuts more than 50 m
Line-to-line distances, km			
0 - 20	0.25 - 0.50	0.10 - 0.25	0.05 - 0.10
20 - 50	0.50 - 0.75	0.25 - 0.50	0.10 - 0.15
50 - 100	0.75 - 1.0	0.50 - 0.75	0.15 - 0.20
100 - 200	1.0 – 1.+25	0.75 - 1.0	0.20 - 0.25
200 - 500	1.25 - 2.0	1.0 - 1.25	0.25 - 0.50

The detail of the measurement also depends on the scale of the map being created. Average values of tack distances of 20, 50, 100, 250 and 500 m are established for scales 1:2000, 1:5000, 1:10000; 1:25,000 and 1:50,000 respectively. The maximum allowable distance between lines is 2 cm on the map scale, the minimum is 0.5 cm, and the average is 1 cm.

The geodetic substantiation of the survey of the water area is a special network created along the coast, based on the points of the state network. There are various methods for creating a planned justification - triangulation, polygonometry, trilateration, straight and resections and their combinations. Justification points are fixed with concrete monoliths and marked on the ground with signals, pyramids or stone tours.

At small distances from the coast, within the coastal zone (up to 10 km), the position of the measurement points is determined by a direct geodetic notch with two theodolites. At a significant distance from the coast of the measuring vessel, its position is determined by radio geodetic systems (RGS).

When planning surveying work, it is necessary to pay attention to the fact that the angles of intersection of directions from the reference points (in a straight geodetic notch) or the angles of intersection of isophase hyperbolas when working with CGS lie in the range from 30 to 150 °.

If the planned position of the vessel is determined by a direct geodetic notch, the survey lines are laid in the form of straight lines parallel to the coast, and when using the CGS, the lines are laid in the form of arcs of circles connected to each other. Tacking of tacks is carried out to the corners of the bases of towers, separate piles, milestones, the coordinates of which are known with an error of no more than 1 m. 2 m

Direct measurements during the measurement process are carried out with a basting and a hand lot at depths up to 5 and 20 m, respectively. The accuracy of measurements depends on the type of soil, the presence of vegetation, currents, waves, and is characterized by errors of 5–10 cm at depths up to 5 m and 10–20 cm at greater depths. Echo sounders are used to indirectly measure depths. An echo sounder is a device that measures depth by measuring the propagation time of an acoustic signal from a vessel to the bottom of a reservoir and back.

The depth of the reservoir h is defined as the sum of the depth $h_{1\ of}$ the reservoir under the emitting AND and receiving P antennas of the echo sounder and the deepening of these antennas Δ from the surface of the reservoir.

The depth of the reservoir is determined by the formula

$$h = 0{,}5\sqrt{(c \cdot t)^2 - l^2}\ , (122)$$

where ℓ is the distance between the transmitting and receiving antennas;

where c is the speed of sound propagation in water, t is the time it takes the sound pulse to travel the distance to the bottom of the reservoir and back.

In most modern echo sounders, the receiving and transmitting antennas are combined and therefore the formula is used to determine the depth

$h = 0.5\ s\ t + \Delta$, where Δ is the antenna immersion depth.

The speed of sound propagation in water varies within

c = 1466 - 1548 m/s, its value is influenced by temperature, depth and salinity of water. The appearance of air bubbles in the water (under the influence of wind and waves, the movement of the vessel at high speed) can cause interruptions in the operation of the echo sounder or errors in its readings.

In the case of sounding work, in addition to the depth, it is necessary to determine the rectangular coordinates (x , y) of the measurement point. These determinations are carried out using a GPS receiver installed on the ship, and at small distances from the coast, an electronic tachometer. An electronic tachometer is installed on the shore, and a reflector is installed on the ship.

The echo sounder allows you to get the profile of the bottom in the direction of the vessel. To survey a given water area, several lines are laid with the required frequency between the lines.

When examining shallow water areas with a complex bottom topography, more detailed information about the situation under water is required. In these cases, multibeam echo sounders are used.

Structurally, such systems can be implemented; in the form of a set of several single-beam echo sounders spaced apart in the horizontal plane and simultaneously measuring vertical distances to the bottom at several points; in the form of a scanning echo sounder, measuring sequentially from the position of its antenna to the bottom in various directions at known angles, which makes it possible to calculate the depth not only under the watercraft, but also to the side, at distances 2–3 times greater than this depth; in the form of several echo sounders, the antennas of which are located at one point, but fixed at different angles to the vertical, which allows you to simultaneously measure the distance to the bottom and calculate the depth at a number of points under the vessel and at distances 2-3 times greater than this depth.

Simrad manufactures the SM 2000 P profiling scanning multibeam sonar Multibeam Profiling Sonar , which includes a scanning antenna combined with an underwater electronic unit and a ship's PC. Scanning occurs in a 120° sector in 128 directions with a scanning beam width of 1.5° or 3°.

Echo sounders are characterized by: range of action - shallow (for depths h < 500 m), mid-depth (h < 5000 m), deep-water (h up to 10,000 - 12,000 m); the number of operating ranges for measuring depths - single-range and multi -range; the number of operating frequencies - single-frequency and multi-frequency; the number of generated probing beams - single-beam and multibeam; the width of the directivity characteristic (XH) of acoustic antennas - narrow-beam (XH < 10°), wide-beam (HH > 10°); type of presentation and method of recording the results of measurements - analog with a recorder and digital with a CCG - (digital depth indicators); spatial orientation of the XH axis - echo sounders with stabilized radiation and echo sounders with unstabilized radiation.

A typical structural diagram of an echo sounder includes emitting, receiving and measuring paths, registration and indication units, control and synchronization devices.

The radiating path consists of a probing acoustic signal generator and a transmitting antenna. *The probing signal is* a periodic sequence of rectangular pulses with tone filling, the power and duration of which are set in accordance with the measured depth. The operating frequencies of acoustic radiation are selected taking into account the depth of the device, the required resolution, the level and spectrum of acoustic interference (in modern echo sounders, the frequency of filling pulses is usually 10 - 300 kHz). The time duration of the pulses is usually equal to 10^{-2} - 10^{-5} s and varies under the condition of providing the necessary resolution in all measurement ranges. Their repetition rate, as a rule, does not exceed 500 pulses/s in the shallow water depth measurement mode and decreases to several pulses per second as the range of measured depths increases to their maximum values.

In table. 25 shows the main characteristics of some echo sounders of the NEL-M series.

Echo sounder characteristics

Table 25

Characteristic	NEL - M1	NEL - M2	NEL - M3A	NEL - M4
Measurement limits, m	1 - 6000	1 - 3000	0.2 - 200	0.2 - 36
Max speed carrier, nodes	35	thirty	40	25
Hardware average quadratic error, m: according to the recorder at h < 20 h >20 by digital depth gauge at h < 20 for h > 20	0.1 _ 1.5 10 -2 – 0.07 10-2 _ h	0.1 - 0.07 10-2 _ h	0.1 1.5 10 -2 – 0.07 10-2 _ h	0.1 - 0.07 10-2 _ h
Permissible roll, degree: keel onboard	3 ten	3 ten	3 ten	3 ten
Number of frequencies	2 (HF, LF)	2 (HF, LF)	1(HF)	1(HF)
Estimated speed of sound, m/s	1460 - 1530	1460 - 1530	1500	1500

Ground surveying is an important part of surveying water areas. It is performed for the display on topographic maps of the lithological type, areal distribution and other characteristics of bottom sediments. During the construction of offshore structures, great importance is attached to the reliability of the geological situation displayed on the maps of water areas.

It is recommended to collect the necessary geological information by taking bottom samples and by sonar survey using side-scan sonar, low-frequency echo sounders and profilers. Aerial photographs and satellite images are used to survey shallow waters.

Detection and determination of the location of underwater pipelines, communication cables, wellheads, sunken ships and other objects of artificial origin is possible by optical, photo-television, hydroacoustic and electromagnetic methods.

If water transparency allows, the first two methods are used. Hydroacoustic and electromagnetic methods are more reliable. A searcher for underwater pipelines IPT has been developed, based on the induction method of search and detection. The device operates at depths up to 10 m and allows you to determine the planned and vertical position of pipeline points with an error of ~ 2 m relative to the carrier.

Level observations. Globally, observations of sea level fluctuations are carried out by an international body called the Permanent Mean Sea Level Service. The data bank of the service contains series of monthly and annual averages for more than 1,000 sites, of which 389 sites have been observed for more than 20 years, and 112 since the last century.

In the waters of the seas and oceans of Russia, level measurements are carried out by the State Committee for Hydrometeorology. In the Baltic Sea, level measurements began in 1703 by decree of Peter I. Regular observations have been carried out in the military harbor of Kronstadt since 1804. Since 1825, observations have been processed to obtain the altitude position of the average level.

Work on the study and forecasting of level fluctuations in the coastal zone and the open sea is carried out at level posts, the hardware basis of which is level gauges. According to the purpose and duration of action, level posts are divided into permanent, additional and temporary; at the place of installation - to coastal and offshore posts; according to the type of measuring and recording devices - on rack, pile and posts with recorders. Posts on the high seas, according to the method of storing and transmitting measurement information, are divided into autonomous and telemetric.

When topographic surveying of water areas, level observations are performed in order to identify current level fluctuations during the survey, take them into account when determining the height of the instantaneous working level of the water surface in the area of work and bring the results of sounding measurements to a single zero of depths or heights. In the cases provided for by the technical designs, level observations during the survey can also be carried out to determine the height of the lowest theoretical level, accumulate data on the long-term average level, transmit absolute height to islands and other coastal objects. The amplitude of the tides depends on the configuration of the coastline and the depths in the coastal zone.

The maximum tide for the World Ocean is 16.2 m, observed at the top of Fundi Bay (Nova Scotia). In the open ocean near islands with deep coasts, the average tide is 0.8 - 1.0 m. For the water areas of the USSR shelf, the maximum tide is 13.0 m near Astronomichesky Cape in the Penzhinskaya Bay of the Sea of \u200b\u200bOkhotsk. Off the coast of the Arctic Ocean, tides do not exceed 0.8 - 1.2 m, in the Baltic Sea - 3-5 cm.

The high-altitude reference network used in Russia for sea level measurements includes a fundamental benchmark, the main benchmark of the level post, tied by leveling I or II class to the fundamental benchmark, a control (working) benchmark, connected by leveling IV class to the main benchmark, and a level rail tied leveling of class IV to the working benchmark. Such a scheme for constructing a network in conjunction with a binding technique makes it possible, from processing 25-year observations, to obtain a water level value with a root-mean-square error related to a one-year observation period of 0.5 mm.

The surface of many Russian seas is covered with ice for 8-10 months, therefore, sounding work on them is carried out from the ice surface using echo sounders installed on all-terrain vehicles. To do this, holes are drilled in the ice with the help of drilling rigs.

In addition to depth measurements, when studying the shelf, geophysical surveys (magnetic, gravimetric, seismic) are carried out, sea currents are measured, and soil samples are taken.

The main scale of topographic maps of the shelf is 1:25,000, the areas of exploration and development of mineral deposits are filmed at a scale of 1:10,000, 1:5,000. Maps of a scale of 1:50,000 are compiled for hard-to-reach areas of the Arctic seas.

Elements of aerial photography.

Currently, topographic maps, plans of scales 1:500 - 1:25000 are created mainly by methods of aerial phototopographic survey. Small-scale maps are compiled using the available large-scale maps using the cameral method.

Aerial photography is carried out using AFA aerial cameras installed on board the aircraft. On the basis of photographs of the area and a previously created survey justification, maps and plans are obtained. Images are processed using photogrammetry methods - a science that determines the shape and size of an object from its photographic image. The survey substantiation is a network of terrain points depicted in the images, the coordinates of which are known (or determined).

Continuous aerial photography is usually used to map large areas. When surveying pipeline routes, aerial photography of the strip of terrain along the selected direction is carried out. After processing the obtained images on photogrammetric devices, a topographic plan of the terrain strip is obtained, which is used to clarify the position of the pipeline route.

Certain requirements are imposed on aircraft for aerial photography: they must be equipped with photo hatches; have special navigational equipment; provide the required photographing height and flight speed; the ability to take off and land on unpaved runways (and not just concrete ones); have a low operating cost.

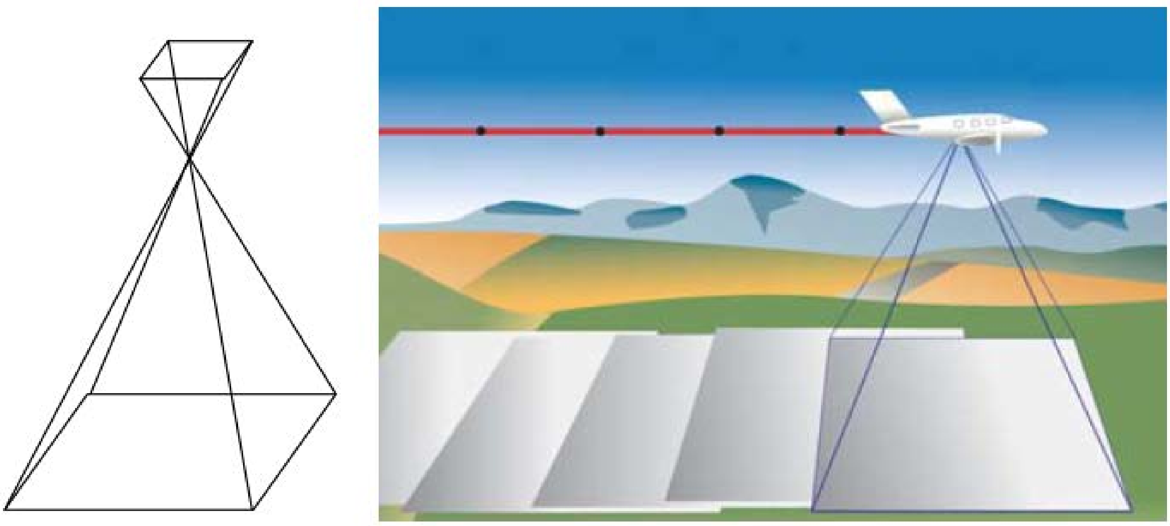


Fig . 74 . The principle of creating a picture using aircraft.

Photographic equipment . Three aerial cameras (AFA) are installed on board the aircraft. Two aerial cameras provide continuous shooting. At the moment of recharging the first one, the second device works. The third backup camera is switched on in case of failure of the first or second aerial camera.

The set of an aerial camera (AFA) usually includes: the AFA itself, removable film cassettes, a set of lenses with different focal lengths, and a command device. The AFA itself is installed on a gyro-stabilizing installation. In addition, the aircraft has a statoscope and a radio altimeter with a photo recorder on board.

AFA scheme. (Fig. 74)

1 – AFA lens with projection center *S* and focal length f_k; 2 - Corps AFA; 3 - Cassette with film (frame 18 x 18 cm).

AFAs with a focal length f_k = 70 mm are most often used.

In addition to AFA, the following devices are used in aerial photography:

The statoscope is a differential barometer for determining the excess between images (for the first time, a differential barometer was developed by D. I. Mendeleev). The accuracy of determining the elevations of neighboring images - 1.9 - 1.4 m

Radio altimeter - a device (pulse rangefinder) for determining the vertical distance from the aircraft to the ground at the time of shooting. Accuracy 1 - 2 m.

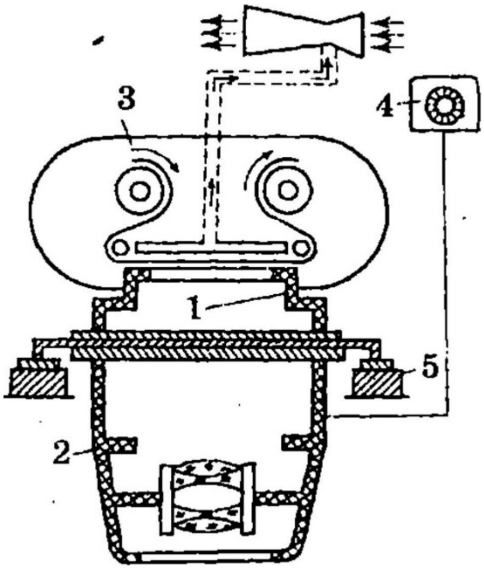


Fig.75. Camera device.

At the time of photographing, fiducial marks are displayed on the image by which you can determine the position of the main point of the image.

At the moment of photographing, the optical axis of the AFA must be vertical. To keep the AFA in the required position, a gyro-stabilizing installation is used that keeps the AFA axis in a vertical position with an error of the order of 30'. Pictures taken with the vertical position of the optical axis of the AFA are called planned or horizontal. In practice, these images are obtained with small (up to 3°) inclination angles. Pictures taken at inclination angles of more than ^{30} are called perspective. The operation of the AFA is controlled by a command device that prepares the frame, exposes and sets the interval for photographing the images. A distinction is made between aerial photography by routes (for the purpose of finding the routes of linear structures) and aerial photography of areas.

When photographing the area along the route, images are obtained at certain intervals, calculated in such a way that adjacent images of the route (Fig. 74) overlap with each other.

To photograph a large area of the area, a number of parallel routes are laid with mutual overlap between them.

For stereoscopic observation of images, it is necessary that the images in the route overlap by at least 60%. Such an overlap is called longitudinal and is denoted by R_x. The overlapping of images of two adjacent strips is called transverse P_y and is necessary for linking routes. Its value is 30 - 40%.

The color of an image is an important deciphering feature. On color photographs, the elements of the terrain and objects located on it are depicted in colors close to natural ones. On multispectral images, terrain elements and other objects are depicted in conditional colors. Images of the same objects in different images may have significant differences in color, since it depends on the conditions of aerial photography, the properties of photographic materials and their processing.

ratios are of great importance in a spectral zonal image , especially when deciphering vegetation, a small hydrographic network, etc.

The shadows of objects and their images in the image play a significant role in recognizing volume objects of small size and contrast. The shadow can be used to judge the shape and height of an object. Some of them (power line supports, antenna masts, etc.) are often recognized only by the shadow. There are own and falling shadows. *Own shadow* is called the unlit part of the surface of the object, located on the side opposite to the Sun. In single shots, own shadows create a visual appearance of the object's three-dimensionality. A *falling* shadow is a shadow cast by an object on the earth's surface. For deciphering, drop shadows are of particular importance. They repeat the shape of the object depending on the position of the Sun and its height. The shape of the shadow allows you to mentally restore the shape of the object, and by the size of the shadow you can determine its height.

Indirect signs are divided into natural, anthropogenic and natural-anthropogenic. *Natural features* include the interconnections and interdependence of objects and phenomena in nature. They are also called *landscape.* Such features can be, for example, the connection of the relief with the geological structure of the area and their joint role in soil formation, in the development of the type of vegetation. The objects used in the identification and determination of the characteristics of objects that cannot be deciphered directly are called *indicators* , and deciphering is called *indicative.* Such deciphering can be multistage, when the direct indicators of decipherable objects are identified with the help of auxiliary indicators. Methods of indication decoding solve the problems of detecting objects that are not displayed on the images. So, using indicator plants, they search for groundwater, minerals, or identify morphostructures, in many cases corresponding to anticlinal or synclinal folds, by the development of a hydro network.

According to *anthropogenic features* , objects created by man are identified . At the same time, functional connections between objects, their position in the general complex of structures are used. For example, points of the reference geodetic network are recognized on images by the allotment of land allocated for them, the type of the point and its shadow. In the forest, sighting clearings are cut through to them, converging at the location of the point. Similarly, agricultural repair shops are identified by the image of the machines located on the territory. Power lines and communications are identified by unplowed plots of land near them. In forest areas, they pass along straight clearings.

In connection with the dependence of human activity on certain natural conditions, *natural-anthropogenic features include manifestations of the properties of natural objects in human activity.* Thus, according to the distribution of certain types of agricultural crops, one can make a judgment about the properties of soils, or according to the presence and location of oilfield facilities - about the shape and size of oil deposits, etc.

The main task of geological interpretation of aerial photographs is the compilation of a high-quality geological map on their basis in a shorter time than by conventional field methods. This is achieved by establishing the boundaries of the distribution of stratigraphic complexes, represented by lithologically heterogeneous strata, using aerial photographs. This is especially clearly revealed in geologically open areas using color spectrum-zonal aerial photography.

The structure of a photographic image on an aerial photograph in a number of cases makes it possible to reveal geological regularities that are difficult to fix in ground studies. Discontinuous disturbances are most easily deciphered on aerial photographs. In the simplest case, it seems possible to determine the rupture of layers by their displacement in amplitude, depicted on the scale of an aerial photograph. The presence of a rupture is detected by the discontinuity of the layers being deciphered, which are expressed by a sharp change in rocks with different tones, a change in relief forms; for shearing layering, pinching out individual horizons, etc. Large cracks are clearly expressed on aerial photographs. Many discontinuities are expressed in the pattern of the hydrographic network. Layering is transmitted on aerial photographs by alternating tones of different densities, depending on the color of these layers. Horizontally occurring layers with a dissected relief give a complex pattern of layer outcrops that repeats the relief forms. With an inclined occurrence and a weakly dissected relief, the layers are elongated along their common strike. Normal anticlinal folds are deciphered by the dip of the layers on the wings, directed in opposite directions from the axial line of the fold.

Features of satellite images, their topographical and geological interpretation.

Space photography currently plays the main role in the complex of remote methods for studying the earth's surface and the upper part of the earth's crust. In terms of the completeness and volume of information contained in one frame, no other type of remote recording can be compared with space photographs. This is explained by a number of circumstances.

A space photograph, by analogy with an aerial photograph, is a familiar type of information for a wide range of specialists. So, in state geological mapping, the use of aerial photographs is considered mandatory. A space photograph containing images of various landscape elements provides material for research by cartographers, topographers, geomorphologists, geographers, and so on. From space photographs taken with metric systems, it is possible to measure all three coordinates and thereby determine various landscape elements and carry out mapping, as well as perform georeferencing of data received by non-photographic receivers.

Space photographs, due to their visibility, significantly complement aerial photographs. Images obtained from space not only have some advantages or disadvantages compared to aerial photographs, but have fundamentally new qualities. This is primarily due to increased visibility and generalization of the image. One satellite image covers an area of several tens of thousands of square kilometers on the earth's surface. It became possible to interpret the results of interpretation in a different way when objects of large sizes, photographed under the same natural conditions by one system, fall into the field of view. Photographic surveys from space, in comparison with aerial photography, provide a significant generalization of the obtained data, which eliminates the masking effect of relatively small objects and landscape elements that interferes with interpretation. It is these possibilities that make it possible to reveal the most general regularities from satellite images, which usually cannot be revealed when using other types of surveys. Finally, the comparison of space images of different times of the same territory makes it possible to identify changes occurring on the earth's surface in connection with exogenous and endogenous processes.

The completeness and volume of information obtained from space photographs depend on their resolution, which is determined by the illumination of landscapes, their brightness, spectral characteristics, optical properties of the lens, technical characteristics of photographic films, and the shooting scale. Modern equipment makes it possible to obtain images with a resolution of 30 m on the ground, and of strongly contrasting objects - up to 5-10 m or less. Photographing from spacecraft is carried out with cameras of the format from 24 to 70 mm with a focal length of 38 to 3000 mm from an altitude of 200 to 400 km. The image scale has a wide range.

Geometrically, satellite photographs differ significantly from aerial photographs. They have large distortions due to the spherical surface of the Earth, changes in flight altitude, and refraction of the atmosphere. At the same time, there are practically no distortions due to the relief in satellite images. Despite the fact that the principles of space photography are in many respects similar to aerial photography, the installation of photographic equipment on spacecraft contributes to obtaining qualitatively and quantitatively new data. Thus, during the use of satellite images, significant experience has already been accumulated, which is used in regional and detailed studies. The foundations of the methodology for deciphering and interpreting satellite images in combination with topographic, geological, geophysical, geomorphological, geochemical, landscape and other materials have been created.

The introduction of satellite survey methods, in particular, made it possible to obtain fundamentally new information about the structural plan of the sedimentary cover, the use of which increases the efficiency of geological and geophysical work and geological prospecting drilling. Remote information contributes to increasing the efficiency of research, accelerating scientific and technological progress. The developed methods for deciphering and interpreting remote sensing materials make it possible to obtain complementary or clarifying ideas about the relief, about the structural plan of the sedimentary cover and the earth's crust as a whole, about the direction, intensity , and degree of differentiation of recent and modern movements. Special studies are aimed at identifying local search objects, searching for building materials, studying permafrost, geothermal anomalies, soils, choosing pipeline and road routes, construction areas , environmental protection, etc.

Cosmogeological interpretation methods made it possible, in particular, to speed up the compilation of survey and regional geological maps, gave fundamentally new information about the development of linear structural elements (faults, fracture zones, flexures) and areal objects of various shapes (polygonal, rounded, etc.), which, in combination with geological and geophysical materials contributes to the - refinement of the structure of the sedimentary cover, tracing zones of favorable prospecting structures.

Owing to the noted properties of space photographs, the landscape-forming role of the geological - structure of the locality stands out in them especially clearly. Therefore, the study of the relationship of geological objects with the components and elements of the landscape is of particular importance when deciphering. The relief is the main indicator of the geological structure of the area. When interpreting large-scale aerial photographs, individual micro- and mesoforms of relief are used as indicators , and when working with small-scale aerial photographs, relief types are also used. Relief microforms are usually not depicted on satellite images and therefore do not play a role. On the contrary, the role of relief types, which can serve as an indicator of the composition of rocks and large lithological-formational complexes, increases significantly. The nature of the boundaries of individual types of relief indicates the relationship of different strata (normal stratigraphic, unconformable, or tectonic contact).

When using interpretation features based on relative excesses, the following circumstance should be taken into account . The minimum excess Δ h , which is caught by the observer's eye during stereoscopic viewing of photographs, depends on the height H flight, basis b shooting and optical zoom ν of the device:

$$\Delta h = \frac{250\,\alpha H}{b\nu}, \qquad (123)$$

where α is the acuity of stereoscopic vision; 250 mm is the distance of the best vision.

Taking α \u003d 30 "(in radian measure - 0.000 145), b = 28 mm, H = 200 km, ν= 1.3 (mirror-lens stereoscope), we get h = 200 m, i.e. elevations up to 200 m are indistinguishable. If the optical magnification of the device is ν ≈ 9 (at a higher magnification, the image usually blurs), the excess will be about 30 m. with lower relative heights can be identified by indirect signs: the nature of the hydro network, vegetation, phototone.

Satellite photographs, which have a resolution an order of magnitude lower than an aerial photograph, depict elements of higher orders. At the same time, high-order watercourses depicted on satellite images make it possible to identify the largest, regional landscape and geological objects: faults, folded structures, and main lithological-formational complexes. The elements of the hydraulic network that are clearly expressed in the relief stand out most confidently on space photographs. In this case, some watercourses can be identified even when, due to their small size, they themselves are not recognized. The manifestation of elements of the hydro network is also facilitated by such a factor as the confinement of vegetation to them, which determines the dark tone of the photographic image.

Unlike ordinary geological maps depicting a section of geological formations on the Earth's surface, satellite images reflect geological objects and phenomena of various depths.

On satellite images at a scale of 1:2500,000 and smaller, platform, folded, orogenic areas are clearly distinguished, deep faults and shifts, as well as zones of transverse faults crossing geosynclinal folded systems, are clearly traced. In a number of areas, buried Proterozoic, Caledonian, and Hercynian folded zones are traced, activated in the modern era, and ancient arched uplifts appear.

On satellite images at a scale of 1:1000000 - 1:500000 transregional and regional systems of lineaments associated with faults of various genesis, swells and zones of local folds are deciphered, and on satellite images at a scale of 1:200000 at the stage of regional work it is possible to identify local landscape elements, which correspond both to known buried folds and to recent structural complications of various scales. In almost all the studied regions, satellite and aerial photographs can be used to identify as many new promising objects as established by geological and geophysical methods. Along the faults traced from the images, zones of wedging out of terrigenous reservoirs of oil and gas complexes can also be localized.

When providing topographic and geodetic support for geological and geophysical work, when planning volumes and placing a network of seismic profiles, effective schemes of aerospace surveys are often used, which show the location of heterogeneous elements deciphered in the images and interpreted as a manifestation of fault and plicative tectonics.

The following terminology is used to unify various linear and areal forms decoded on satellite images by tonality, structure, or color. Linear elements are divided into two groups: 1) rectilinear (lineaments), with subgroups by size - global, transregional, regional, zonal, local; 2) curvilinear (rounded, annular, concentric, oval, arc). Areal elements are classified according to the shape of the boundaries: polygonal, rounded (ring, oval), irregular shape.

As a result of regional and detailed aerospace studies , it was established that the confirmation of photoanomalies as local oil prospecting objects is 90%.

Space and aerial photographs play an important role not only in the search for and development of new deposits, but also in the restoration of old oil fields, inventory of the field economy , and are also an effective means of clarifying the position of old communication lines, pipelines, etc.

Thus, space images and space maps have a multi-purpose application, reflect objects and phenomena of various depths and, in particular, are becoming increasingly important in predicting promising provinces and oil and gas accumulation zones, and are used in the oil industry.

The geometric features of satellite images noted above determine the distinctive methods for solving various kinds of problems of measurement interpretation from space photographs, which are usually classified into three groups.

The first group includes measurements made directly from a space photograph. In this case, one of the essential features of photogrammetric processing is the need to take into account the influence of the curvature of the earth's surface. Within certain limits, this influence may turn out to be insignificant, then it is possible to use the known methods of aerial photogrammetry discussed above. Therefore, the first task is to establish the boundaries within which the earth's surface can be considered a plane.

The second group involves transferring the results of interpretation to the map base and carrying out measurements using the usual methods and methods of cartography. Identification of a photogrammetric image of the earth's surface with a map is possible within certain limits, since a space photograph and a map are generally built in different projections. However, sometimes the space photograph itself is considered as a photographic map of a certain cartographic projection. Thus, the main task in comparing a space photograph with a map is to find out the permissible boundaries of their identification.

The third group is a set of problems solved by analytical methods. In this case, the sought values can be found from the image coordinates, such as the areas of deciphered contours, distances, etc. As a rule, this problem of measurement interpretation can be solved using a computer, which requires the creation of appropriate programs. The basis of such problems is a direct photogrammetric resection, when using the known elements of external orientation, the flat coordinates of the image are converted into spatial coordinates of the terrain.

Chapter 11
Brief information from the theory of errors

§ 51. Gross, systematic and random measurement errors.

Distinguish between *necessary measurements and redundant measurements.* So, if the same value is measured n times, then one of these measurements is necessary, and o steel (n - 1) are redundant. Redundant measurements are needed to control the measurement results, as well as to obtain a more reliable value of the measured quantity. In addition, the presence of a sufficient number of redundant measurements makes it possible to assess the accuracy of the measurements performed.

The error of the measurement result arises as a result of the action of many factors. Errors arising from the action of individual factors are called *elementary.* The error of the measurement result is made up of elementary errors. According to the nature of the impact on the results of measurements, errors are gross, random and systematic .

Gross errors or blunders arise as a result of the inattention of the performer; they are detected during repeated measurements and the results containing gross errors are excluded from processing. For example, at the technical leveling station, the excess is determined twice by the black and red scales of the rails. Divergence of excesses h_h and h_{to} no more than 5 mm is allowed. If this condition is not met, the results are rejected, and the excess is determined again with a different level horizon.

Systematic errors arise if a permanent factor is not taken into account; they affect the measurement results with one sign or according to a certain law. A systematic error is sought to be eliminated from the measurement results by introducing a correction or by using an appropriate measurement technique. However, it is not always possible to completely eliminate the systematic error. As an example of the elimination of a systematic error, one can cite the introduction of a correction for the length of the measured line for comparating the instrument.

It is believed that it is impossible to completely eliminate systematic errors , and they are included in the measurement results as elementary ones. random errors.

Random or inevitable errors are those errors whose magnitude and sign change randomly. Collectively, random errors obey one or another probabilistic law, called *the law of distribution of random errors.* More often than others, when processing geodetic measurements, the law of normal distribution (Gauss's law) is used.

Errors in determining the increments of coordinates.

The coordinates of the points obtained in accordance with expression (1) are approximate, which creates uncertainty in determining the spatial position of the well. This uncertainty is due to:

errors in modeling the trajectory of the well axis, or the choice of an approximating function based on discrete measurements

measurement errors of angular and linear parameters in wells.

On the segment between the measurement points of the well parameters, its trajectory can be approximated either with the help of a tangent or with the help of a chord.

In the first case, the segment between adjacent measurement points over the entire interval l is considered as a tangent to the circular arc at the measurement point, i.e. spread the values of the angles θ and αmeasured at this point over the entire interval.

In the second variant, the segment l is considered a chord, while the angles θ and αmeasured at the end points of the interval are averaged. This option is used most often.

Coordinate error due to measurement errors.

To determine the error of coordinates due to measurement errors, we use the formula for the error of a general function $y = f(x_1, x_2, ..., x_n)$, where $x_1, x_2, ..., x_n$ are independently measured values with root mean square errors $m_1, m_2, ..., m_n$. The root mean square error of a function of this kind will be

$$m_y^2 = \left(\frac{\partial f}{\partial x_1}\right)^2 \cdot m_1^2 + \left(\frac{\partial f}{\partial x_2}\right)^2 \cdot m_2^2 + ... + \left(\frac{\partial f}{\partial x_n}\right)^2 \cdot m_n^2 \quad (124)$$

We take partial derivatives of expressions (1) with respect to variables l_i, θ_i and α_i we get

$$m_{x_K}^2 = \sum_{i=1}^{K} \sin^2\theta_i \cdot \cos^2\alpha_i \cdot m_l^2 + \sum_{i=0}^{K} l_i^2 \cdot \cos^2\theta_i \cdot \cos^2\alpha_i \cdot m_{\theta_i}^2 +$$
$$+\sum_{i=0}^{K} l_i^2 \cdot \sin^2\theta_i \cdot \sin^2\alpha_i \cdot m_{\alpha_i}^2$$
$$m_{y_K}^2 = \sum_{i=1}^{K} \sin^2\theta_i \cdot \sin^2\alpha_i \cdot m_l^2 + \sum_{i=0}^{K} l_i^2 \cdot \cos^2\theta_i \cdot \sin^2\alpha_i \cdot m_{\theta_i}^2 + \quad (125)$$
$$+\sum_{i=0}^{K} l_i^2 \cdot \sin^2\theta_i \cdot \cos^2\alpha_i \cdot m_{\alpha_i}^2$$
$$m_{z_K}^2 = \sum_{i=0}^{K} l_i^2 \cdot \sin^2\theta_i \cdot m_{\theta_i}^2 + \sum_{i=1}^{K} \cos^2\theta_i \cdot m_{l_i}^2$$

where m_{θ_i} and m_{α_i} expressed in radian measure $\left(m_\theta = \frac{m_\theta}{\rho}; m_\alpha = \frac{m_\alpha}{\rho} \right)$

The length measurement error l is $\frac{m_l}{l} = \frac{1}{1000}$ much less than the measurement errors θ and α, therefore, the term containing the formulas can be neglected and then expressions (2) can be written

$$m_{x_K}^2 = \sum_{i=1}^{K} l_i^2 \cdot \cos^2\theta_i \cdot \cos^2\alpha_i \cdot m_{\theta_i}^2 + \sum_{i=1}^{K} l_i^2 \cdot \sin^2\theta_i \cdot \sin^2\alpha_i \cdot m_{\alpha_i}^2$$
$$m_{y_K}^2 = \sum_{i=1}^{K} l_i^2 \cdot \cos^2\theta_i \cdot \sin^2\alpha_i \cdot m_{\theta_i}^2 + \sum_{i=1}^{K} l_i^2 \cdot \sin^2\theta_i \cdot \cos^2\alpha_i \cdot m_{\alpha_i}^2 \quad (126)$$
$$m_{z_K}^2 = \sum_{i=1}^{K} l_i^2 \cdot \sin^2\theta_i \cdot m_{\theta_i}^2$$

Example : $l_1 = l_2 = \ldots = l_k\ , \quad k = 10$

(well depth ~1000m)

$\theta = 30^\circ; \quad \alpha = 45^\circ; \quad m_\theta = 0,5^\circ; \quad m_\alpha = 3^\circ$

$при \ \ \alpha = 45^\circ; \quad m_x = m_y$

$$m_{x_K}^2 = m_{y_K}^2 = 10 \cdot 100 \cdot 100 \cdot \left(0,75 \cdot 0,5 \cdot 0,000076\right) +$$
$$+10 \cdot 100 \cdot 100 \cdot \left(0,25 \cdot 0,5 \cdot 0,00274\right) = 37,10$$
$$m_{x_K} = m_{y_K} = 6м$$

линейное смещение будет в $\sqrt{2}$ *раз больше, т.е* $8,5м.$

$$m_{z_K}^2 = 10 \cdot 100 \cdot 100 \cdot 0,25 \cdot \frac{1}{13133} = 1,9$$
$$m_{z_K} = 1,38м$$

To improve the accuracy of calculating the coordinates of the points of the well axis, the formulas are sometimes used

$$x_K = x_0 + \sum_{i=0}^{K} l_i \cdot \sin\frac{\theta_{i-1}+\theta_i}{2} \cdot \cos\frac{\alpha_{i-1}+\alpha_i}{2}$$
$$y_K = y_0 + \sum_{i=0}^{K} l_i \cdot \sin\frac{\theta_{i-1}+\theta_i}{2} \cdot \cos\frac{\alpha_{i-1}+\alpha_i}{2} \quad (127)$$
$$z_K = z_0 - \sum_{i=0}^{K} l_i \cdot \cos\frac{\theta_{i-1}+\theta_i}{2}$$

in which the θ_{i-1} , θ_i , α_{i-1} , α_i zenith and directional angles measured at the beginning and end of the interval l_i .

§ 52. Property of random errors. Arithmetic mean of measurement results . Mean quadratic, limiting and relative errors. Medium quadratic error of the measured values. Root mean square error arithmetic mean

Measurement is the process of comparing some physical quantity with a homogeneous quantity, taken as a unit of measurement. The result of any measurement q is a number (or coefficient) showing how many times the measured value Q is greater or less than the unit of measurement τ:

$$Q = q \times \tau \ . \ (128)$$

Measurements are direct (immediate) and indirect (mediocre). *In direct measurements* , the quantity to be determined is directly compared with the unit of measurement. (For example, when measuring the length of a segment with a tape measure).

In *indirect measurements* , the quantity to be determined is obtained after measurements of other quantities functionally related to the quantity to be determined. (For example, to calculate the elevation using the trigonometric leveling formula, it is necessary to measure the angle of inclination, distance , instrument height and target height).

When carrying out measurements, it is necessary to solve the following questions: with what accuracy to measure and by what methods (and instruments), how to assess the accuracy of the results obtained, and how to avoid gross miscalculations.

The measurement result inevitably contains an error. If we denote by *l* the result of measurements, and by X the exact value of the measured quantity, then the difference between them Δ is called the true error

Δ \ u003d *l* - X. (129)

The exact value of the quantity being measured is usually unknown, but in some cases it can be taken as the measurement results obtained by a more accurate instrument or method.

Sometimes the value of the measured value can be found from mathematical calculations, patterns. For example, if angles are measured in a polygon, then it is known in advance that the sum of these angles will be 180 ° (n - 2).

Measurements are performed under the following conditions and factors : measurement object, observer, measuring device, measurement method, environment, measurement moment.

Measurements made under the same conditions are called *equivalent measurements.* Due to changes that occur with the object of measurement, the observer, the device and the environment over time, the measurement conditions vary and the measurement results cannot be considered equal in these cases. However, it is generally accepted that in case of compliance with the requirements and rules set forth in regulatory documents, geodetic measurements can provide equally accurate results. (For example, when measuring elevations during technical leveling, when measuring angles in a theodolite traverse).

The main properties of random errors are as follows.

1 . For given measurement conditions, random errors cannot exceed a known limit.

2. Positive and negative errors, equal in absolute value, occur equally often in a series of measurements.

3. Smaller in absolute value random errors are more common than large ones.

4. The arithmetic mean of random errors tends to zero as the number of measurements increases.

$$\lim_{n\to\infty}\frac{\Delta_1+\Delta_2+\Delta_3+........+\Delta_n}{n}=\frac{[\Delta]}{n}=0 \qquad (130)$$

Average

Let for some quantity, the true value of which is X , n measurements are made and the measurement results are obtained ℓ_1, ℓ_2, ℓ_3 ………., ℓ_n.

You can find the true errors for this type of measurement.

Δ_1 \ u003d ℓ_1 - X

Δ_2 \ u003d ℓ_2 - X (131)

Δ_3 \ u003d ℓ_3 - X

…………..

Δ_n \ u003d ℓ_n - X

We add the left and right parts of these equalities

$[\Delta] = [\ \ell\ ] - nX$. (132)

In accordance with the fourth property of random measurement errors

$[\ \Delta] = 0$ as n tends to infinity. Therefore, one can write

$\lim X = \frac{[\Delta]}{n}$ for $n \to \infty$

With a finite number of measurements, the arithmetic mean of a series of measurements always differs from the true value by some value ε

$X = \frac{[\Delta]}{n} - \varepsilon$ (133)

In this case, the resulting arithmetic mean is denoted *X*

$x = \frac{[\Delta]}{n}$, where x = X + ε (134)

Assessment of the accuracy of measurement results

The accuracy of a number of measurements can be assessed by various parameters. Let's consider some of them.

1. Average error Θ . It is calculated as the arithmetic mean of the absolute values of the true random errors

$$\Theta = \frac{[|\Delta|]}{n} \qquad (135)$$

2. Probable error r . If the absolute values of measurement errors are arranged in descending or ascending order, then the error located in the middle of this series will be the probable error. Those. this is the error, more and less than which the absolute value of the error is equally probable. If two errors turn out to be average in the series, then the probable error of the series will be the average of their absolute values.

3. The mean square error m .

The concept was introduced by K.F. Gauss. The formula for calculating the root mean square error is as follows

$$m = \sqrt{\frac{[\Delta^2]}{n}}, \text{ where } [\Delta^2] = \Delta^2{}_1 + \Delta^2{}_2 + \Delta^2{}_3 + ... + \Delta^2{}_n \qquad (136)$$

Compared to the previous accuracy criteria, it is more efficient and stable. Those. the root mean square is sensitive to *large* random ones and changes little with a change in the number of measurements.

Example. Let two series of random measurement errors be given

I row: -1 +4 -3 +1 -2 +1 0 -4 +3 -2

II row: -2 -1 +7 -2 +1 0 -5 0 +3 0

We calculate the average errors $\Theta_{I} = 2.1$; $\Theta_{II} = 2.1$ - it turns out that the measurement results are equally accurate.

We calculate probable errors $r_{I} = 2$; $r_{2} = 1.5$ it turns out that the measurement results of the second row are more accurate than the first.

We calculate the mean square errors

$m_{I} = 2.5$; $m_{II} = 3.0$ which show that the results of the first row are more accurate than the second. Indeed, in the second row , there is a greater spread in the error values.

4. Limiting error. In the course of geodetic work, it is very important to timely detect measurement results containing gross errors in order to eliminate them from further processing and replace them with better results.

The experience of a large number of measurements shows that in a number of random errors, only 5 out of 100 errors can exceed the double rms error and only 3 out of 1000 errors can exceed the triple rms error. Therefore, as a marginal error Δ prev. take on a value

Δprev . = 2 m or Δ prev. = 3m

For example, the root-mean-square error of determining the excesses on the 1 кмprogress of technical leveling 25 мм, and the limiting 50 мм. Therefore, the maximum allowable discrepancy for exceeding the course of technical leveling is calculated by the formula $f_{h\,доп} = 50мм\sqrt{L}$, where L is the length of the course in km.

When laying a theodolite traverse, the root-mean-square error of measuring one angle with the T-30 theodolite is 30", and the limit is taken to be 1 ´ .5.

All measurements with errors greater than Δ prev. discarded as gross and repeated anew.

5. Absolute and relative errors.

According to the form of expression, the errors are divided into absolute and relative. Absolute errors are expressed in the same units of measurement as the measured value, therefore errors: average, probable, root mean square, limit - absolute errors.

Relative error f rel. - a number showing the ratio of the absolute error to the value of the measured value L.

Relative error is used to assess the accuracy of measuring distances, areas, volumes. Absolute error is used to evaluate the accuracy of angle measurements.

Root mean square error of the function of the measured values.

So far we have dealt with the estimation of the accuracy of directly measured quantities. Often the quantity being determined is a function of other directly measurable quantities. In this case, the question arises of determining the root-mean-square error of the function of the measured quantities. For example, the base *a* and the height *h are known* triangles measured with mean square errors m_a and m_h*;* find the mean square error of the area of the triangle m_S.

In probability theory, a formula is known for calculating the root mean square error of a function of independently measured quantities. Let the function y \ u003d f (x_1 , x_2 , ..., x_n) , (137)

where x_1 , x_2 , …, x_n — independently measured quantities; their variances will be equal to σ_1^2 , σ_2^2 , …, σ_n^2 . The variance of a function of this kind will be

$$\sigma_y^2 = \left(\frac{\partial f}{\partial x_1}\right)^2 \sigma_1^2 + \left(\frac{\partial f}{\partial x_2}\right)^2 \sigma_2^2 + \ldots + \left(\frac{\partial f}{\partial x_n}\right)^2 \sigma_n^2 . \qquad (138)$$

In practice, instead of variances, the squares of root-mean-square errors are used, and instead of formula (1), the formula is used

$$m_y^2 = \left(\frac{\partial f}{\partial x_1}\right)^2 m_1^2 + \left(\frac{\partial f}{\partial x_2}\right)^2 m_2^2 + \ldots + \left(\frac{\partial f}{\partial x_n}\right)^2 m_n^2 \qquad (139\ )$$

Let's return to the example with the calculation of the mean square error of the area of a triangle. The area of a triangle S is determined by the well-known formula

$$S = \frac{1}{2} ah \qquad (140)$$

We use the formula (139). The mean square error of the area will be determined by the expression

$$m_s = \sqrt{\left(\frac{1}{2}h\right)^2 m_a^2 + \left(\frac{1}{2}a\right)^2 m_h^2} .$$

The root mean square error of the arithmetic mean.

In the case of equally accurate measurements, the arithmetic mean is calculated using the formula

$$\bar{x} = \frac{[l]}{n} , \qquad (141\ )$$

where [*l*] is the sum of the measurement results; n *is* their number.

Expression (3) can be written differently -

$$\bar{x} = \frac{1}{n} l_1 + \frac{1}{n} l_2 + \ldots + \frac{1}{n} l_n . \qquad (142)$$

The root mean square error of a function of this kind will be

$$m_x^2 = \left(\frac{1}{n} m_1\right)^2 + \left(\frac{1}{n} m_2\right)^2 + \ldots + \left(\frac{1}{n} m_n\right)^2 . \qquad (143)$$

Considering that the measurements are equally accurate - m_1 \ u003d m_2 \u003d ... \ u003d m_n \ u003d m , we get

$$m_x^2 = \frac{m^2}{n}, \qquad (144)$$

or otherwise

$$M = m_{\bar{x}} = \frac{m}{\sqrt{n}}. \qquad (145)$$

The standard error of the arithmetic mean to the square root of the number of measurements less than the standard error of any of the results from which the arithmetic mean is derived.

Bessel formula

When calculating the root mean square error by the formula

$$m = \sqrt{\frac{[\Delta^2]}{n}} \qquad (146)$$

it is assumed that the exact value of the measured quantity is known , and random errors are found using the formulas

Δ_1 \ *u003d* l_1 - X ;

Δ_2 \ *u003d* l_2 - X ; (147)

……....

Δ_n \ u003d l_n - X.

Often the value of the measured quantity is unknown, and the arithmetic mean is used as it $\bar{x}$.

In this case, the deviations v_i - of the measurement results from the arithmetic mean are calculated (sometimes v_i - are called probable errors):

v_1 \ *u003d* l_1 - X ;

v_2 \ *u003d* l_2 - X ;

…….... (148)

vn _ = *n* _ - X .

After summing the left and right sides of equality (148), we obtain

$[v] = [l] - n\bar{x}$.

But earlier it was said that $\bar{x} = \frac{[l]}{n}$ or $n\bar{x} = [l]$, therefore $[v]$=0.

The sum of deviations from the arithmetic mean is always zero.

We subtract term by term from (147) equality (148), we get:

$\Delta_1 - v_1 = \bar{x} - X$;

$\Delta_2 - v_2 = \bar{x} - X$;

………………

$\Delta_n - v_n = \bar{x} - X$.

Earlier it was noted that ($\bar{x}$ - X) is some small value ε, therefore

$\Delta_1 = v_1 + \varepsilon$;

$\Delta_2 = v_2 + \varepsilon$;

…………..

$\Delta_n = v_n + \varepsilon$.

After squaring both parts of these equalities and adding them, we get

$$[\Delta^2] = [v^2] + n\varepsilon^2 + 2\varepsilon[v].$$

Since $[\nu]$=0, we can write

$$[\Delta^2] = [\nu^2] + n\varepsilon^2$$

or otherwise

$$\frac{[\Delta^2]}{n} = \frac{[\nu^2]}{n} + \varepsilon^2 . \qquad (149\ )$$

Can be replaced $\frac{[\Delta^2]}{n} = m^2$; the value ε is unknown and it can be replaced by the root mean square error of the arithmetic mean -

$$\varepsilon = M = \frac{m}{\sqrt{n}} .$$

Therefore, the expression (149) can be written as follows:

$$m^2 = \frac{[\nu^2]}{n} + \frac{m^2}{n} \text{ or}$$

$$m^2 n - m^2 = [\nu^2].$$

Therefore, we get $m = \sqrt{\frac{[\nu^2]}{n-1}}$.

This is the Bessel formula. We note here that as the number of dimensions increases, the Bessel formula is identical to the Gauss formula.

Estimation of Accuracy by Differences of Double Equal Accurate Measurements.

In the practice of geodetic measurements, to exclude rough processes, the same value is measured at least twice. Assume that each value of each row is measured twice and all measurements are equally accurate (for example, the values of horizontal angles obtained from two half-takes), it is required to determine the root mean square error of one measurement. So, there are a number of double equal measurements l'_1, l''_1 ; l'_2, l''_2 ;...; l'_n, l''_n .

Let's find the differences

$d_1 = l'_1 - l''_1$;

d_2 \ *u003d* $l'_2 - l''_2$;

…………

$d_n = l'_n - l''_n$;

If the measurements were accurate, then the differences between the double measurements would be zero, so we can write

Δ_{d_1} \ u003d d_1 -0 \u003d d_1 ;

Δd_2 \ u003d d_{2-0} \u003d d_2 ;

…………

Δ dn \ u003d d n -0 \u003d d n .

The root-mean-square error of one difference is obtained by the formula

$$m_d = \sqrt{\frac{[d^2]}{n}} . \qquad (150)$$

Difference d_i is a function of two equal measurements -

d_i \ *u003d* $l_i' - l_i''$.

Therefore, $m_d = m\sqrt{2}$, where m is the root mean square error of one measurement, i.e.

$$m = \frac{m_d}{\sqrt{2}} . \qquad (151)$$

Now we substitute here the value m_d from the expression (151) and get

$$m = \sqrt{\frac{[d^2]}{2n}} . \qquad (152\ )$$

The root mean square error of the average of the two measurements will be equal to

$$m_{cp} = \frac{m}{\sqrt{2}} \text{ or } m_{cp} = \sqrt{\frac{[d^2]}{4n}} . \qquad (\ 153)$$

Formulas (152) and (153) can be used to calculate root mean square errors if the differences d no systematic errors. If there is a systematic error in the measurements, it can be found by the formula

$$\theta = \frac{[d]}{n} .$$

In the case when θ deviates from zero by a negligibly small amount , there is no systematic error, but if θ ≠ 0, it is necessary to exclude the value of θ from the differences in double measurements. You calculate the residuals

$$d_i' = d_i - \theta .$$

Residual differences d_i' are similar to the most probable errors $[d']$=0, so the root mean square errors can be calculated from the formulas

$$m = \sqrt{\frac{[d'^2]}{2(n-1)}} ;\ m_{cp} = \sqrt{\frac{[d'^2]}{2(n-1)}} . \qquad (154)$$

§ 53. Expression of the root mean square error in terms of the most probable .

So far, we have considered equal-precision measurements, i.e. measurements performed under the same conditions. However, in practice, very often measurements are not performed under the same conditions and they correspond to different standard errors. Therefore, such measurements are called unequal measurements. Unequal measurements have different reliability, different degrees of confidence.

Weight is the reliability of the measurement result, expressed as a number. The more reliable the result, the greater its weight. Thus, the weight is related to the accuracy of the measurement result, which is determined by the root mean square error. The weight of the measurement result is inversely proportional to the square of the root mean square error and is determined by the formula

$$p \backslash u003d\ k / m^2, \quad (155)$$

where k is a constant value (proportionality factor, which is introduced to facilitate calculations); *m is the* mean square measurement error.

If there are results of unequal measurements $l_1, l_2, \ldots, l_n$ and their root-mean-square errors $t_1, t_2, \ldots, t_p$, we can calculate the weights of these measurements

$$p1\ \_ = k / t_1{}^2;\ p_2 = kmm_2{}^2; \ldots;\ p_n \backslash u003d\ k / m_n{}^2. \quad (156)$$

For example, let we have the results of measurements l_1 and l_2 and their root mean square errors m_1 =4 and m_2 = 12. Let us calculate the weights of these results

$$p1\ \_ = k / 16;\ p2\ \_ = k / 144. \quad (157)$$

Let 's take k = 144, then p_1 = 9, $p2$ = 1 .

One could take k = 16, then p_1 = 1; p_2 = 1/9, i.e. the ratio of the weights does not change. It is important to note here that the results of equally accurate measurements have equal weights.

Arithmetic mean weight

The weight of the arithmetic mean P can be determined by the formula

$$P \backslash u003d\ k / M^2, \quad (158)$$

where *M is* the root mean square error of the arithmetic mean. Recall that

$$M = \frac{m}{\sqrt{n}},\ , \quad (159)$$

where m is the root mean square error of a single result of equally accurate measurements, therefore, we can write

$$P = \frac{kn}{m^2}.,\quad (160)$$

But in turn $k / t^2 = p$ is the weight of one measurement, which can be taken as a unit - p = 1. Thus, we get $P = n.$ Therefore , with equally accurate measurements, the weight of the arithmetic mean is equal to the number of measurements from which it was obtained.

Root mean square error per unit of weight

If we take the weight of the result of any measurement (in the series of results of unequal measurements) equal to one, and denote by μ its mean square error, then we can write

$$1 = k / \mu^2 \text{ or } \mu^2 = k. \quad (161)$$

The general expression for the weight in this case can be represented as

$$p_i = \mu^2 / m_i{}^2. \quad (162)$$

μ value is called the root mean square error of unity weight.

Weight average

Suppose that there are results of unequal measurements of the same value l_1, l_2, ..., l_n and the weight of these results p_1, p_2, …, p_n. Each of the results can be represented as the arithmetic mean of p_i equal measurements:

$$l_1 = \frac{[l]_1}{p_1};\ l_2 = \frac{[l]_2}{p_2};\ldots;\ l_n = \frac{[l]_n}{p_n}. \qquad (163)$$

The arithmetic mean of all measurement results will be equal to

$$L = \frac{[l]_1 + [l]_2 + \ldots + [l]_n}{p_1 + p_2 + \ldots + p_n}. \qquad (164)$$

We can write $[l]_1 = l_1 p_1$, $[l]_2 = l_2 p_2$,…, $[l]_n = n\_p_n$, hence

$$L = \frac{l_1 p_1 + l_2 p_2 + \ldots + l_n p_n}{p_1 + p_2 + \ldots + p_n} \text{ or } L = \frac{[lp]}{[p]}. \qquad (165)$$

The weight of the weight average is equal to the sum of the weights of the measurement results from which it was obtained $P = [p]$.

If through M_o denote the root mean square error of the weight average, then from formula (10) we can write

$$P = \frac{\mu^2}{M_0^2} \text{ or } M_0 = \frac{\mu}{\sqrt{P}}. \qquad (166)$$

Considering that R = [p], we get $M_o = \dfrac{\mu}{\sqrt{[p]}}$. (167)

The root mean square error of the weighted mean to the square root of the sum of the weights is less than the root mean square error of the result, the weight of which is equal to one.

Examples of processing in assessing the accuracy of measurement results.

Example 1. Theodolite T30 is used to measure horizontal angles in a theodolite traverse. The root-mean-square error of measuring the horizontal angle with one complete reception with the TZO theodolite is equal to m_{in} \u003d 30 ".

It is required to determine the allowable discrepancy (limiting error) in the sum of the angles of a closed theodolite traverse.

The sum of the angles of the traverse -

$$\sum_{1}^{n} \beta = \beta_1 + \beta_2 + ... + \beta_n, \qquad (168)$$

where n is the number of corners in the theodolite traverse.

As a result of applying formula (168), we write

$$m^2_{\sum_1^n \beta} = m_1^2 + m_2^2 + ... + m_n^2. \qquad (169)$$

Considering that *m 1 \ u003d m 2 \u003d ... m n =* m_β (equivalent measurements), we obtain

$$m^2_{\sum_1^n \beta} = m^2{}_\beta n \text{ or } m_{\sum_1^n \beta} = m_\beta \sqrt{n}. \qquad (170)$$

Let's take Δ pre =3 *m* , then Δ pre =3 $m_\beta \sqrt{n}$. In angular measurements it is customary to designate the marginal error as the allowable discrepancy Δprev = $f\beta_{доп}$; here we recall that *m in* \u003d 30", then

$$f\beta_{доп} = 1{,}5' \sqrt{n}. \qquad (171)$$

Example 2. The results of equally accurate measurements of the same excess are given (Table ...). It is necessary to find the arithmetic mean , the mean square error of one measurement, and the mean square error of the arithmetic mean.

Calculate the arithmetic mean using the formula

$$\bar{x} = l_0 + \frac{[\varepsilon]}{n}, \qquad (172)$$

Processing the results of equal measurements

Table 26

Measurement number	Measurement result l , mm	ε , mm	v, mm	v ^{2}
one	1043	+06	+03	9
2	1040	+03	00	0
3	1039	+02	-01	one
four	1039	+02	-01	one
5	1037	00	-03	9
6	1041	+04	+01	one
7	1042	+05	+02	four
	l_0 = 1037	$[v]$=00	$[v^2]$=28	$[\varepsilon]$=+21

where l_0 - conditional average; ε - deviation from the conditional average; $\bar{x}$ = 1037 + 21/7 = 1040.

Next, we find deviations from the arithmetic mean using the formula $v_i = l_i - \bar{x}$.

We calculate v_i^2 and $[v^2]$. RMS measurement error will be equal to

$$m = \sqrt{\frac{[v^2]}{n-1}} = \sqrt{\frac{28}{6}} = 2{,}1 \text{ mm}.$$

The mean square error of the arithmetic mean -

$$M = \frac{m}{\sqrt{n}} = \frac{2{,}1}{\sqrt{7}} \approx 1\text{mm}$$

Example 3. Below are the values of the differences in the results of double measurements of the sides of the traverse. It is necessary to calculate the standard error of one measurement, the standard error of the line value of the average of two measurements, and the relative standard error of the average if the lengths of the sides of the traverse are approximately 200 m.

Line number........... 1-2 2-3 3-4 4-5 5-6

d .... +15 -10 +12 -14 +03

d2 .... 225 100 144 196 9

Line number........... 6-7 7-8 8-9 *n* = 8

d . ... -05 +07 -08 $[d]$=00

d^2 .... 25 49 64 $[d^2]$= 812 The sum of the differences *d* is equal to zero, so we can assume that the results measurements do not contain systematic errors. Using formula (8), we find the root mean square error of one measurement

$$m = \sqrt{\frac{812}{2\times 8}} = 7{,}1\text{see \_}$$

The root mean square error of the mean of two measurements is

$$m_{\text{ср}} = \frac{7{,}1}{\sqrt{2}} = 5{,}0\text{ see \_}$$

The relative root mean square error of the mean is calculated as follows

$$\frac{m_{\text{ср}}}{\text{S}} = \frac{0{,}05\text{м}}{200\text{м}} = \frac{1}{4000}.$$

Example 4. When performing a tacheometric survey, the excess points are determined by the formula

$$h = \frac{1}{2}D\sin 2\nu + a - l,$$

where the value *D* determined by a filament rangefinder; ν - angle of inclination (measured along the vertical circle of theodolite); *a* - theodolite height above the standing point; *l* - target height. Find the root mean square error in determining the excess.

The errors in measuring the height of the theodolite and the height of the sighting target do not exceed 1 cm, and they can be ignored. Using formula (169), we obtain

$$m_h^2 = \left(\frac{dh}{dD}\right)^2 m_D^2 + \left(\frac{dh}{d\nu}\right)^2 \frac{m_\nu^2}{\rho^2} \text{(173)}$$

or

$$m_h^2 = \left(\frac{1}{2}\sin 2\nu\right)^2 m_D^2 + \left(D\cos 2\nu\right)\frac{m_\nu^2}{\rho^2}. \qquad (174)$$

We accept for these conditions m_ν = 30", m_D / D = 1/300, D = 100 м, ν = 3°. Then by formula (3) we get: m_h = 2,2 см, and the limit error $\Delta_h = 2m$ = 4,4 см.

Chapter 12

§ 54. Leveling methods

Leveling is called work in order to determine the difference in heights of points on the earth's surface, as well as their heights relative to the accepted reference surface. As a result of leveling, the excesses *h are determined* , and then the heights of the points *H are calculated* .

Types of leveling

Geometric leveling is performed by a horizontal sighting beam, which is obtained using devices called levels.

Trigonometric leveling is done with an inclined beam. To determine the excess, the angle of inclination and the distance between points are measured. Applied instruments: theodolites and tacheometers.

Physical leveling is divided into barometric, hydrostatic and air leveling.

Geometric leveling is the most common type of leveling.

To determine elevations , a horizontal line of sight is used here, created by a geodetic instrument - a level.

To perform the work, leveling rails are also required, which are installed at the leveled points of the terrain. The position of the horizontal sighting beam is fixed by readings on vertically standing rails.

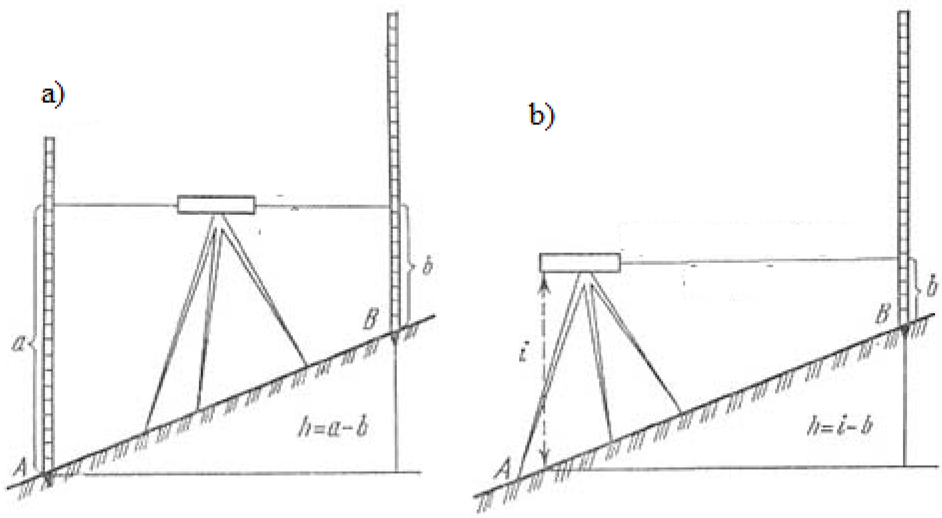


Fig.76. Geometric leveling a) from the middle; b) forward.

Two methods of geometric leveling are used: “from the middle” and “forward”.

When leveling “from the middle” (Fig. 76, a), the level is set so that the distances to both points are the same, and when leveling “forward”, the device is installed (Fig. 76, b) near one of the points. The excess

h in both cases is determined by the formula

$$H \backslash\, u003d\; a - in\,, \qquad (175)$$

where a is the reading on the leveling staff installed at point A ;

c - reading along the rail installed at point B. the excess h has a plus sign when $a > b$ and a minus sign when $a < b$.

If the height of point A is known, then the height of point B is calculated by the formula

$$H_B \backslash\, u003d\; H_A + h\,. \qquad (176)$$

When leveling “from the middle” in the direction from point A to point B , the rail installed at point A is called the back, and the rail at point B is called the front.

Leveling "from the middle" is used when laying leveling moves. The advantage of this method is that it compensates for errors associated with non-fulfillment of the basic geometric condition of the level (the main geometric condition of the level is that the line of sight must be horizontal).

The “forward” leveling method is used when it is necessary to determine the heights of a large number of points from one leveling installation. In this case, it is convenient to calculate the marks of points from the horizon of the GP device - the marks of the horizontal line of sight:

$$GP \backslash\, u003d\; H_A + a\,. \qquad (177)$$

The marks of the determined points are found by the formulas:

$$H_B \backslash u003d\; GP - in,$$
$$H_C \backslash u003d\; GP - s,$$
....
....
$$H_N \backslash u003d\; GP - n\,, \qquad (178)$$

where in , s , n - readings on the rails installed at points B , C , ..., N .

§ 55. The principle and methods of geometric leveling .

Sequential leveling, leveling move.

Sequential leveling or leveling move, that is, leveling with several device settings (Fig. 42) is used at a significant (more than 200 m) distance or excess between two points. At points 1, 2, 3 ....., called binders, the rail is installed on wooden stakes, metal pins (crutches) or special metal shoes. The total excess between points A and B is equal to the sum of elementary excesses:

$$h_o \backslash\, u003d\; h_1 + h_2 + h_3 + \ldots \ldots + h_n\,.$$

You can write $h_1 \backslash\, u003d\; a_1 - in_1$,

$h_2 \backslash\, u003d\; a_2 - in_2$,

....

h_n \ u003d a_n - in_n .

Summing up the left and right sides of the equalities, we get:

$$h_o = \sum_{eleven}^{n} a - \sum^{n} in . \qquad (179)$$

The total excess is equal to the sum of the back rail readings minus the sum of the front rail readings.

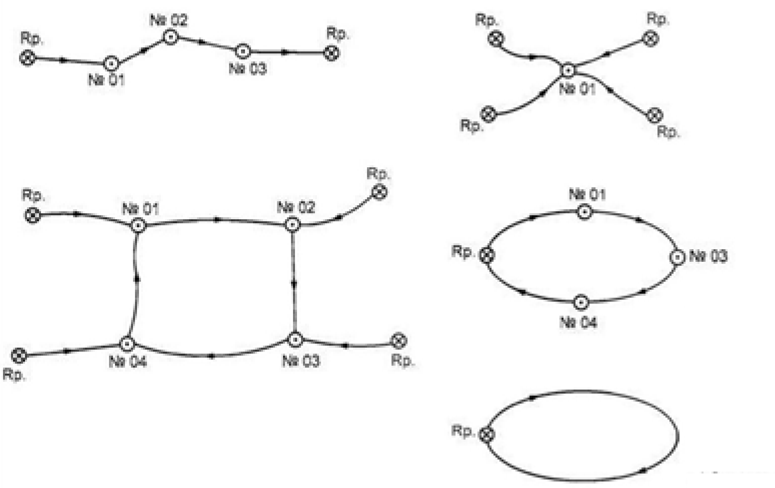


Fig.77. Types of leveling moves.

Closed leveling course. A closed path is a move that starts and ends at the same point. The course is laid (Fig. 77) from the benchmark (R_P) - a point with a known height - to determine the heights of points 1, 2, 3, ... The sum of excesses in a closed course is zero. Therefore, we can write:

$$_{theor}\sum_{one}^{n} h = 0 \qquad (180)$$

The sum of practically obtained excesses will not be equal to zero, but will be equal to some value called the discrepancy f_h ,

$$_{pr}\sum_{one}^{n} h = f_h . \qquad (181)$$

The value of the discrepancy should not exceed the value of $_{additional} f_h$, set depending on the class (accuracy) of leveling.

If $f_h \leq {}_{add} f_h$, then the excesses are corrected by introducing an amendment, the value of which is V_h is determined by the formula

V_h \u003d - f_h / n , (182)

where n is the number of excesses (or stations).

The marks of the points are calculated, starting from the benchmark, according to the formula

N_K \ u003d $H_{K-1} + h_{\text{got stuck}}$, (183)

where N_{K-1} - the mark of the previous point;

h_{stuck} - the excess, corrected by the amendment, between the previous and the determined point.

Open leveling stroke. Open is called a leveling course, which is laid (Fig. 77) between two points with known heights. On fig. 77 the course is laid from the benchmark *A* to the benchmark *B* to determine the heights of points 1, 2, 3 .... The sum of the excesses between the two benchmarks is known. Theoretically, it is equal to the difference between the marks of these benchmarks:

$$_{theor}\sum_{one}^{n} h \backslash \text{ u003d } H_B - H_{A.} \qquad (184)$$

From the measurements, the practical sum of the excesses is obtained

$$_{pr}\sum_{one}^{n} h = h_1 + h_2 + \ldots\ldots + h_n . \qquad (185)$$

The discrepancy in excesses along the course is calculated by the formula

$$f_h = {}_{pr}\sum_{1}^{n} h - {}_{theor}\sum_{one}^{n} h \qquad (186)$$

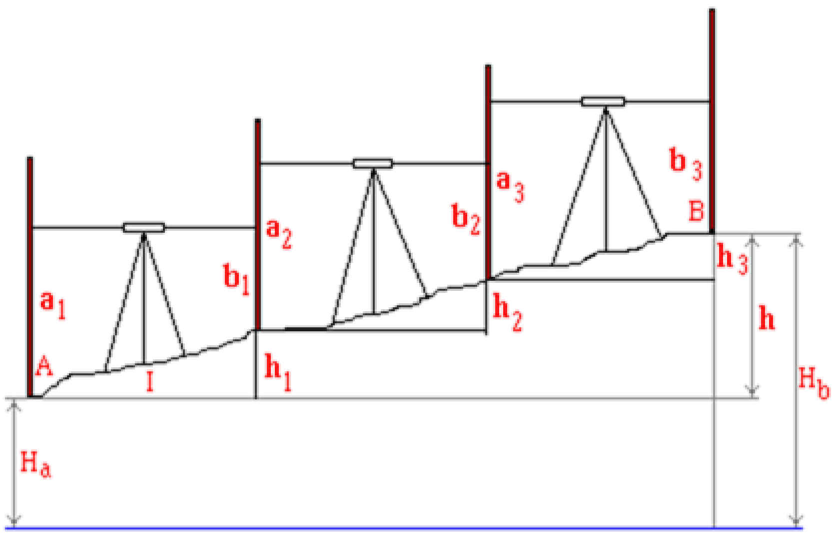
Further, the same operations are performed as in a closed leveling course.

§ 56. Complex leveling.

Regardless of the method, geometric levelling can be simple or sequential (complex). If the elevation between points is obtained as a result of a single installation of the instrument, then it is called simple.

If the purpose of leveling is to transfer elevations over a considerable distance, or to construct a longitudinal profile of the terrain, it is carried out from several stations and is called complex (sequential).

A complex (sequential) leveling creates a level course in which the points are common for 2 adjacent stations and are called tie points. The points where the staff is placed between the tie points are called intermediate points and are usually used to get the marks of the characteristic points of the terrain. Marks of intermediate points are usually calculated through the horizon of the instrument:



Field log processing.

Field log processing includes determining the elevations between tie points, checking the calculations page by page, linking the elevations, and calculating the tie and intermediate point elevations.

Before the beginning of calculations in the field journal the missing data according to the number of the variant of the task is written out: marks of the initial reference points 7 and 8, readout on the marks at the stations 9 and 10 and the picket designation of the end of the trace.

Calculate exceedances between tie points using readings on the black and red sides of the rear and front rails: hhh=ah-hh, hcr=akr-hcr

Perform a page-by-page check of the calculations, which consists in checking the equality (∑a-∑v)/2=∑h/2=hcf.

Discrepancies of 1 -2 mm may occur due to rounding the values of average exceedances to a whole number of mm.

Note: readings from the rods at intermediate points are not used in page control.

Calculate the height discrepancy of the course by the formula fh=∑hcr-∑hteor=∑hcr-(Hkon-Nach), where Hkon=Hnk8, Hnach-Hnk7 - marks, respectively, of the end and beginning points of the course.

Actual height discrepancy should not exceed permissible, equal to fhdop=50mm*√L, where L - stroke length, km.

In my example: Fhdop= 50mm*√1,1 = 55 Fn=∑Hpr-∑theor= -5126+5081=-45

Since fn<fdop, the actual misalignment is distributed with opposite sign equally on all exceedances. Correction for each excess δh= -fn/n=45/15=3 , where n - number of stations in the course.

The corrections are calculated with rounding to mm and are signed in red ink under the values of average exceedances; the sum of corrections should be equal to the misalignment with the opposite sign, i.e. ∑δh=-fn

Calculate the corrected (coordinated) exceedances khispr=hi+δh control: ∑hispr = ∑hteor= Nkon - Nach

The corrected exceedances are used to calculate the tie point marks Hnk0=Hrn7+h1spr...Hrn8=Hnk10+35+h15spr.

The control of correctness of calculations of marks of connecting points is reception of known mark of an end point of movement.

Calculate marks of intermediate points of a course through horizon of device GP for this purpose at station twice calculate GP concerning back and forward binding points of its two values define average. For example for station 12 we have GP'12=Hpk8+a12; GP"=Hpk9+v12; GP12=(GP'12+GP"12)/2

§ 57. The device of the level.

A level is a geodetic device that creates a horizontal sight line in space, relative to which the elevations are determined (for geometric leveling).

By design, levels are optical and digital. According to the method of bringing the sighting axis of the level to a horizontal position, there are levels with a level and levels with a compensator. A separate group consists of laser levels.

Accuracy levels are divided in accordance with the data in table 8.

Table 28

Type of level	a brief description of
H - 05	A high-precision level with an optical micrometer for determining elevations with a root-mean-square error of not more than 0.5 mm per 1 km of a double course
N - 3 (N - ZL, N - ZK, N - ZKL)	Accurate level for determining elevations with a mean square error of no more than 3 mm per 1 km of a double course
N - 10 (N - 10L, N - K, N - 10KL)	Technical level for determining elevations with a mean square error of not more than 10 mm per 1 km of a double course

Optical levels are produced with a level at the telescope and with a tilt angle compensator. The letter K means a compensator, the letter L means the presence of a limb for measuring horizontal angles.

Level H - Z. Optical level with level

Level H - Z is a level with a cylindrical level. On fig. 45 shows a diagram of the H-Z level. Here 1 is a stand with three lifting screws, 2 is an elevator screw, 3 is a telescope, 4 is a cylindrical level, 5 is a rack, 6 is a fixing screw and 7 is a pipe guide screw. The diagram does not show a round level.

Level N - ZK. Optical level with compensator.

The scheme of the level H - ZK is shown in fig. 46. here 1 is an objective, 2 is a focusing lens, 3 is a movable prism suspended on crossing threads, 4 is a fixed prism, 5 is a grid of threads, 6 is an eyepiece.

The preliminary setting of the level is carried out on a circular level. Prism type compensator. Lens 3 is suspended on crossing threads. There is a damper for vibration damping 7. The operating range of the compensator is ± 15'. Guiding the pipe on the rail is carried out first by turning by hand, and then by the screw of infinite guidance.

The main requirement for levels is that the line of sight must be horizontal. To check the fulfillment of this condition, checks are made and, if necessary, corrections are made - the alignment of the level.

Digital levels

Digital levels differ from optical levels in that the reading along the rails is performed automatically by a reader. On the rail, instead of the usual divisions, a barcode is applied. Simultaneously with the receipt of the reading, the distance to the rail is determined. The devices are equipped with a processor and a program that allows you to calculate the elevation of points, distances to points to be leveled, as well as accumulate field measurement data in the device's memory.

Table 29 lists the main characteristics of some digital levels.

Table29 **Digital Levels**

Specifications	Firm and model		
	Sokkia SDL30M	Trimbe DiNi12 DiNi22	Leica DNA 03 DNA 10
Root mean square measurement error per 1 km double run	0.6	0.3 0.7	0.3 0.9
Distance measurement accuracy	0.1% ×D - 0.2% ×D	0.05% ×D	0.05% ×D
Measurement time, *sec.*	3	3 2	3
Working temperature	-20°- + 50°	-20°- + 50°	-20° - + 50°
Compensator operating range	±15′	±15′	±10′
Device weight , kg	2.4	3.5 3.2	2.8 2.8
Built-in memory	2000 measurements	2200 lines	6000 measurements

Laser levels and reference plane builders

Laser levels are designed to set a horizontal, inclined or vertical plane using a laser beam. The main difference between laser devices and optical levels is the ability to see the constructed reference plane.

These devices are successfully used in high-rise layouts, monitoring the operation of planning machines, installing plumbing equipment, monitoring trench development and laying pipes in a trench at a given slope. Some of the devices allow you to create vertical reference planes or lines, which is used to transfer the axes to the mounting horizons, control the installation of columns, check the verticality and evenness of the walls of various structures.

When controlling the development of a trench and laying pipes along a given slope, a laser slope indicator is used, which is installed in the well fig. 47 . The laser beam is given a design slope. The evenness of the bottom of the trench is controlled by a rail installed on the bottom after the excavator. Noticed deviations are eliminated manually. Pipe laying (see Fig.) is controlled using a screen - a brand installed at the end of the mounted pipe section. Concentric circles are applied on the matte screen, with the center of which, if the pipes are laid correctly, the center of the laser spot should coincide. Laser sights for pipe laying control are made in a hermetic design and have a scale for setting the design slope. As a slope sensor, they use a cylindrical or electronic level with a servo drive, which automatically keeps the laser beam at a given slope.

Spectra - physics ' Dailgrade device for laying underground pipelines has a moisture-proof housing, an electronic level, a slope setting range of 15% to + 40%. The accuracy provided is less than ± 5 mm per 100 m. The range is up to 150 m.

On fig. 48 shows the use of a laser horizontal plane builder to control the operation of planning mechanisms.

Before starting work, the laser builder of the reference plane - transmitter *2* (Fig. 48) is installed in the center of the area being developed and brought into working position. The mark of the reference plane H_{op} created by the transmitter is determined after measuring the height of the beam *h* above the reference *3*, the mark of which H_{rp} is known,

$$H_{op} \backslash u003d H_{rp} + h . \qquad (187)$$

Set the photodetector *1* to the required height, which is calculated by the formula

$$l \backslash u003d N_{op} - N_{pr} , \qquad (188)$$

where N_{pr} is the design mark of the developed area.

From one station, the transmitter can serve several sections (Fig. 78) with different design marks H_{pr1} , N_{pr2} , respectively, for these sections there will be their own values \ *u200b\u200bl* $_1$, l_2 .

During operation, depending on which or which elements of the photodetector are illuminated by the laser beam, one of seven lamps turns on on the indicator panel located in front of the operator of the earth-moving mechanism. Using this lamp, the operator can determine where the mechanism is located: at the design mark, above or below it.

There are two options for working with the system: "manual", when the operator of the mechanism, guided by the indicator readings, manually controls the cutting element of the planning machine, and automatic, when the position of the working element is controlled by a photodetector.

The manual mode is used at the stage of preliminary planning, when the main volumes of soil are moved. The automatic mode is used at the final planning stage. The operating range of such systems is up to 500 m. The standard error of planning is about ±3 cm. Table 10 shows the main characteristics of laser levels and builders of reference lines and planes of various companies.

Table 30 Laser levels and builders of reference lines and planes

Model	Firm	Type of compensator fine/range	Plane	Laser type	Purpose	Accuracy	Action radius

Electronics Level	Spectra-Physics	electronic ±10		laser diode (unseen)	Leveling		
L 130	-		horizon.			4 mm at 1000 m	130 m
L 220	-		horizon.			4 mm at 50 m	220 m
L 500	-		horizon.			2.6 mm at 50 m	500 m
L 750	-		horizon.			2.6 mm at 100m	750 m
Plumb Plain 1146	Spectra-Physics _	electronic	vertical	see.	Builds. work	2.4 mm at 30 m	150 m
Dialgrade 1250	Spectra-Physics	electronic	beam with slope us from -15% to 40%	see.	styling podz.pipe-wires	5 mm at 100 m	150 m
LASERPLANE	SPEKTRA PRECISION	thread with			levels management plan machines		500 m
L 500 C	-	air	horizon.	invisible			600 m
L 600	-	damper	horizon.				800 m
L 800		±8" / ±11′	horizon.	see.	control work plan		
laser plane Model 1145	SPECTRA Physics	5 - 8" / ±5°	inclination along two axes up to 9.99%	see. see. see.	nir.mechan. builder-work	2.4 mm at 30 m	300 m
Lasermat	SPEKTRA PRECISION SOKKIA	±20" / ±6°			construction construction construction	0.1 mm per 1 m	150 m
LP-30	SOKKIA	±10" / ±10′	horizontal and vertical	see.	construction	±1.5 mm on 30 m	300 m
LP-31	NEDO Germany	±15" / ±10′	horizon.	invisible		±2.25 mm on 30m	120 m
PRIMUS		±20" / ± 5 °	horizon. horizontal flat vertical beam	invisible see.		±0.2 mm per 1m	100 m

§ 58. Checks and adjustments of the level.

he different types of Levelling are: 1. Differential Levelling 2. Check Levelling 3. Precise Levelling 4. Reciprocal Levelling 5. Longitudinal Levelling or Longitudinal Sectioning 6. Cross Levelling 7. Levelling for Giving Levels for Works 8. Barometric Levelling 9. Hypsometry 10. Trigonometrical Levelling.

Differential Levelling:

It is carried out with the object of determining the reduced levels of points some distance apart or to establish bench marks. The process has already been described in Continuous or compounds levelling and is also known as running flying levels from one point to the other.

In flying levelling, only the back and fore sights are necessary and the shortest convenient route between the points is selected. The length of a sight is kept as long as the power of the telescope and local obstacles permit.

Check Levelling:

It is conducted for the purpose of checking a series of levels, which have previously been fixed. At the end of each day's work a line of levels starting from the point and returning to the starting point of that day is run with the object of checking the work done on that day.

Since the circuit is completed i.e., the levelling work ends at the starting point, therefore, for the work to be correct the difference between the sum of all the back sights and that of all therefore sights on that day should be zero.

Precise Levelling:

It is special method of levelling used for establishing bench marks with high precision at widely distant points, it is conducted by some govt., agency such as Great Trignometrical Survey of India department for establishing G.T.S. bench marks.

It requires the use of highly refined and modern instruments and greatest care in the field as described below:

Instruments:

A high grade level equipped with tilting screw, stadia wires and coincidence level etc. and an invar precision levelling staff are commonly used.

Precautions while Levelling:

(i) The adjustment of the level are carefully tested.

(ii) The parallax should be entirely eliminated by correct focussing.

(iii) The staff should be exactly vertical. It may be plumbed with the staff-level or plumb-bob.

(iv) The bubble should be exactly in the centre of its run at the time of taking readings.

(v) Lengths of sights are limiting to about 100 m.

(vi) The back sight and fore sight distances should be exactly equal. Stadia readings may be taken for this purpose.

(vii) Ground for level and staff should be stable. To avoid error due to settlement of tripod and staff, the back sights and the following fore sights should be taken in quick successions and the order of taking readings is interchanged at alternate set up i.e. at first setting, the back sight is observed first and then the fore sight while at the 2nd setting, the foresight is taken first and then the back sight and so on.

(viii) Levelling work should be suspended in rainy and windy days and also at noon in hot summer days. If work is necessarily to be done under such conditions, level should be protected from the sun or wind by a screen or umbrella.

(ix) Check levelling should be performed by a different surveyor on different days with different change points. And if the closing error exceeds the permissible value, the work should be repeated.

Reciprocal Levelling:

It is a method of levelling adopted to determine the difference of levels between two points when it is, not possible to set up the level midway between them as in the case of a river, a deep valley etc.

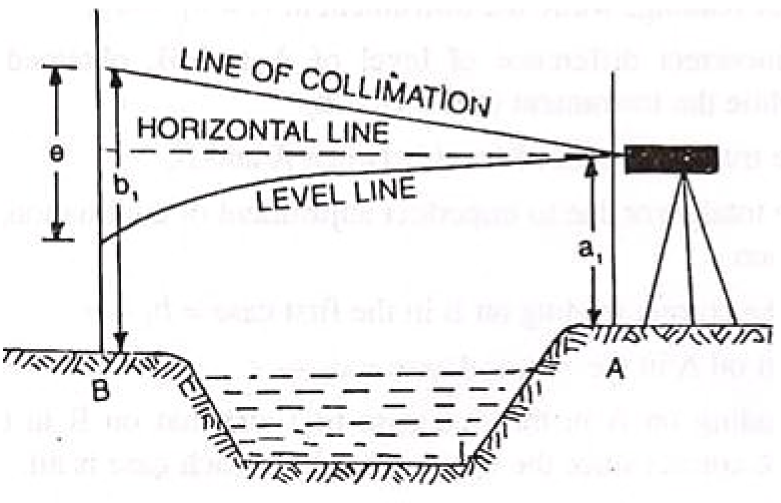
This method also eliminates:

(i) The error due to curvature and refraction, and

(ii) The error due to line of collimation not being parallel to the bubble line which may otherwise occur due to non —equality of back and fore sight distances.

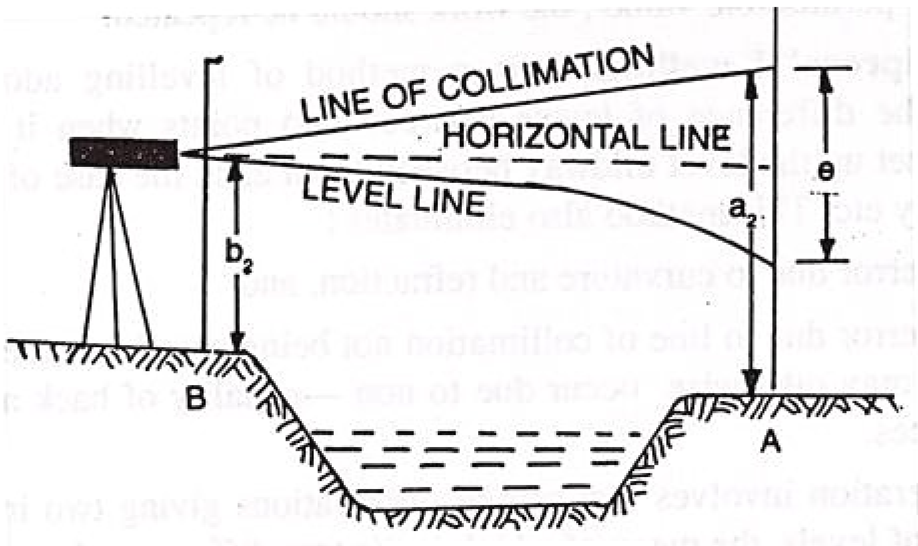
The operation involves two sets of observations giving two incorrect differences of levels, the mean of which is the true difference. Let A and B be two points [Fig- 7.3 (a) and (b)] on opposite banks of a wide river.

Then to find the true difference of level between A and B, proceed as follows:
(i) Set up and level the instrument very near to A or over A [fig. 7.31. (a)].



(ii) Read the staff held at A and B and let the staff readings be (a_1) and (b_1) respectively.

(iii) Transfer the instrument to B and set it up and level very near to B or over B [fig. 7.31 (b)].



(iv) Observe the readings on the staff held at A and B and denote them as (a_2) and (b_2) respectively.
Note:

If the staff is near to the level, then the reading on it should be taken through the object-glass or the height of the centre of the eye-piece above that point should be measured directly with the staff and be considered as staff reading on that point.

Since the combined effect of collimation error, curvature and refraction in the readings is proportional to the distance of the staff from the instrument, the error in reading in both the cases are equal as the lengths of sight in each case is same.

Let in each case the reading at B be greater than that at A i.e., $b_1 > a_1 > b_2 > a_2$.
and let, d_1 = incorrect difference of level of A and B., obtained from the observed readings while the instrument at A = $b_1 — a_1$.

d_2 = incorrect difference of level of A and B, obtained from the readings while the instrument at B = $b_2 – a_2$.

d = the true difference of level between A and B.

e = the total error due to imperfect adjustment of collimation, curvature and refraction.

Then the correct reading on B in the first case = $b_1 — e$
and that on A in the second case = $a_2 — e$
The reading on A in the first case (a_1) and that on B in the second case (b_2) are correct since the length of sight in each case is nil.
In the first case, the true difference, d = correct reading at B —correct reading at A $(b_1 — e)$ a_1

$$= (b_1 - a_1) — e \qquad \text{........}(i)$$

Similarly, in the second case, the true difference, d = correct reading at B —correct reading at A:

$$= b_2 — (a_2 — e)$$

$$:: (b_2 — u_2) + e$$

∴ Adding to eliminate e, we get

$$2d = (b_1 — a_1) + (b_2 — a_2)$$

$$d = \frac{(b_1 - a_1) + (b_2 - a_2)}{2}$$

$$\mathbf{d} = \frac{d_1 + d_2}{2}$$

i.e., the true difference of level between two points equals the mean of the two incorrect differences of level obtained in the two settings of the instruments.

Equating we have

$$(b_1 - a_1) - e = (b_2 - a_2) + e$$

or

$$2e = (b_1 - a_1) - (b_2 - a_2)$$

$$e = \frac{(b_1 - a_1) - (b_2 - a_2)}{2}$$

$$\text{The total error, } e = \frac{d_1 - d_2}{2}$$

Example :

In levelling between two points A and B on opposite banks of a river, the level was set up near A and the staff readings on A and B were 2.156 and 3.568 respectively. The level was then moved and set up near B, and the respective staff readings on A and B were 1.968 and 3.262. Find the true difference of level of A and B.

Solution:

Incorrect difference of level in the first case = 3.56- 2.156= 1.412

Incorrect difference of level in the second case = 3.262- 1.968= 1.294

True difference of level of A and B = mean of the two incorrect differences of level

$$= \frac{1.412 + 1.294}{2}$$

=1.353m (fall from A to B) (Ans).

Longitudinal or Levelling:

The object of this levelling operation is to obtain a record of the undulations of the ground surface along the centre line of a proposed engineering project such as a road, railway or canal etc. The outline of the surface thus obtained is called the longitudinal section or profile so why this levelling is also known as profile levelling. From such a section, an engineer is in a position to study the relationship between the existing ground surface and the proposed formation of the new work in the direction of its length.

The operation involves observing the elevations of a number of points, along the centre line and also their distances along it. The line of section may be a single straight line running in the centre or may consist of a series of straight lines changing direction or connected by curves.

The levels are taken at uniform intervals of distance along the centre line depending upon the requirements of the work and nature of the ground. Besides these points the staff readings are taken at the fairly significant points where outline of the ground changes appreciably so that the profile may be obtained as natural as possible.

Running the Longitudinal Section:

The line of section is set out on the ground and marked with pegs driven at equal intervals usually 20 or 30 metres before starting the levelling operation. The levelling operations always commence at a bench mark and end on a bench mark.

If the permanent bench mark is not available near the line of section, a flying level is run from any permanent bench mark in the vicinity to establish a bench mark near the line of section. The level is set up in such a position as to command as many points on the section as possible.

Staff readings are the taken on pegs fixed already at desired regular intervals and also at significant points of change of slope. These readings are entered in the appropriate columns of the level-book against the respective changing along the line which are recorded in the distance column.

When it is necessary to shift the instrument on account of the length of sight being beyond the power of the telescope (usually 100 m) or the further points not being possible to be observed owing to the irregularities of the ground, a suitable change point is selected on firm ground or a well -defined permanent object and a fore sight is taken.

The change point may or may not lie on the line of section. The level is then shifted and set up in a new commanding position of chaining and observing the intermediate readings is obtained at the last point on the section. If the permanent bench mark exists near the end point of the section line, the work may be closed on it by running flying levels from the last station.

While running a section, it should be kept in view that position of the features such as a nallah, canal, river, road, railway, foot-path, cart-track etc., crossed by the section line are completely located and the arithmetical check is applied to the levelling work at the end.

Checking the Levels:

If the levelling starts from a permanent bench mark and ends on a permanent bench mark the difference between the sums of the back and fore sights must agree with the difference of levels between the first and the final bench mark within permissible limits of the closing error.

And if the levelling work is not closed on the permanent bench mark, the only way to check it is to take flying levels back to the original bench mark and finding the closing error. If the closing error exceeds the permissible value, the work must be repeated.

Plotting the L-Section:

In plotting the longitudinal section, a horizontal line is drawn as datum line and changes of the staff points are marked along this line to a convenient scale. At these plotted points, perpendiculars are erected and on each of these lines, the respective levels are set off. The plotted points are then joined by straight lines to obtain the outlines of the ground surface.

The horizontal scale used in plotting the distances of the points is the same as that of the plan but the vertical scale used in plotting the levels is always different and larger than the horizontal one so as to mark the inequalities of the ground more apparent. The horizontal scales commonly used are 10 m and 25 m to one cm and the vertical scale 1 m or 2 m to 1 cm.

The elevation of the datum line should be so assumed for each separate sheet that the length of the ordinates remain 15 cm. The assumed R.L. of the datum should be written on the datum line and the changes and the R. Ls. of the points are written against the perpendiculars. The datum and the ground lines are drawn in black and the perpendiculars in thin blue lines.

Working Profile:

When the design of an engineering project is made, a working profile is prepared for the use of engineer at site. It shows the features of the original ground surface, the formation levels of new work, the proposed gradient, the depths of cutting and heights of filling and any other information which may be useful during construction of the work.

The new work is represented by thick red line and the original ground by a comparatively thin burnt-sienna line. The natural surface levels are written in black or burnt-sienna and the formation levels in red. The gradients of new work are shown prominently and limits of each clearly shown by arrows. The depths of cutting are written in red and the heights of filling in blue. A road profile worked out from the assumed data.

Table31.

Station	Distance	Bearing		Readings			H of I	R.L.	Remarks
		Back	Fore	B.S.	I.S.	F.S.			
				1.512			269.860	269.348	B.M.1
				1.682		2.368	269.174	267.492	C.P.
				1.778		2.476	268.476	266.698	C.P.
1	0		10530	0.846		1.984	267.338	266.492	C.P.
2	20				1.640			265.698	
3	40				2.784			264.554	
4	60				3.942			263.396	
5	69				0.212			267.126	
6	80			3.836		3.848	267.326	263.490	C.P.
7	100				3.004			264.322	
8	120				2.164			265.162	
9	140				1.310			266.016	
10	146				2.816			264.510	
11	152				3.224			264.102	
12	160			2.368		3.782	265.912	263.544	C.P.
						1.964		263.948	B.M.2

Total = 12.022 16.422

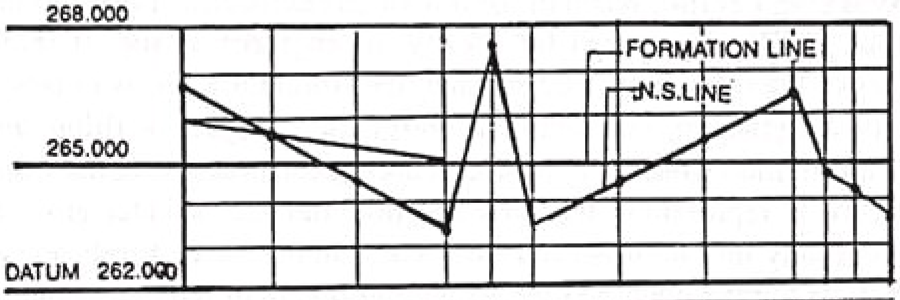
Arithmetical Check : $\Sigma B.S. - \Sigma F.S$ $= 12.022 - 16.422$ $= -4.400$

last R.L – 1st R.L. $= 263.948 - 268.348$ $= -4.400$

Proposed Road Gandhi Park, Jalandhar, R.D. 0 –160

Scales : [Horizontal 1 cm = 10 m]

[Vertical 1 cm = 1m]



FILLING			0.779	1.604		1.510	0.678			0.490	0.898	1.456
CUTTING	0.182	0.031			2.126			0.162	1.016			
FORMATION LEVELS	266.030	265.667		265.000	265.000	265.000	265.000	265.000	265.000	265.000	265.000	265.000
GRADIENT												
N.S. LEVELS	266.492	265.698	264.554	263.396	267.126	263.690	264.322	265.162	266.016	264.510	264.102	263.544
CHAINAGES	0	20	40	60	69	80	100	120	140	146	152	160

Cross Levelling:

The purpose of cross-sectioning is to determine the necessary information regarding the levels of the ground on either side of the longitudinal section for computing the quantities of earth work etc. for designing the engineering projects. Cross-sections are run at right angles to the centre line at 20 or 30 m intervals along the centre line.

The length of the cross-section depends upon the nature of the proposed work. In the case of an ordinary road the length may be 30 to 60 m on either side of the centre line and in the case of a railway, it may vary from 200 to 300 m or even more on either side of the centre line.

Taking Cross-Section:

Cross-sections are serially numbered from the start of the centre line and taken simultaneously with the longitudinal section. They are set out at right angles to the centre line with the chain or tape, the cross-staff, the optical square or the theodolite etc.

The staff is held at each 10m points and other points of sudden change of slope on the cross-section. Readings are then taken from the instrument stations used for the longitudinal section and the distance of the staff points measured with the tape towards left and right of the centre station.

The measurement of the cross-section distances may be entered in the level-book, R or L, written after each to indicate whether the measurement is to the right or left of the centre line. If the longitudinal section measurements are to be entered in the distance column, the cross-distances should be written in the remarks column.

It is sometimes considered preferable to complete the longitudinal section first and then taking the cross-sections so as to ensure correctness of the profile work through necessary checks before taking cross-sectioning in hand. In such a case, the levelling operation for cross-sectioning starts either from the nearest bench mark of from each central peg, the level of which is previously determined during longitudinal sectioning.

In either case, the instrument is set up to command as many cross-sections or points there on as possible and the staff reading taken and recorded as usual. To avoid confusion, the booking of each cross-section should be entered on a separate page of the level –book.

Plotting the Cross-Sections:

The cross-sections are plotted in the same manner as the longitudinal section except that both the horizontal and vertical measurements are plotted to the same scale as is commonly used for the vertical dimensions of the profile i.e., 1 m or 2m to 1 cm.

The R.L. of the datum line may be different for different sections to keep the ordinates fairly short. A cross section for the assumed data at change 60 of the Longitudinal section of the previous problem.

Table 32.

Station	Distance	Bearing		Readings			H.I.	R.L.	Remarks
		back	Fore	B.S.	I.S.	F.S.			
1	0C			1.868			265.284	263.396	0–C is a point on the centre line of the longitudinal section at chainage 60
2	10R				1.346			263.918	
3	20R				0.854			264.410	
4	30R				0.202			265.062	
5	10L				2.612			262.652	
6	16L				1.624			263.640	
7	20L				3.336			261.928	
8	30L				3.856	3.256		261.408	

Arithmetical Check : .Total = 1.868 3.850

ΣB.S.Σ—F.S.
= 1.868 –3.856
= – 1.988

Lat R.L.—Ist, R.L.
=261.408–263.396
= –1.988

Cross-section at chainage—60

Scales : { Hor : 1 cm = 5 m
Ver : 1 m

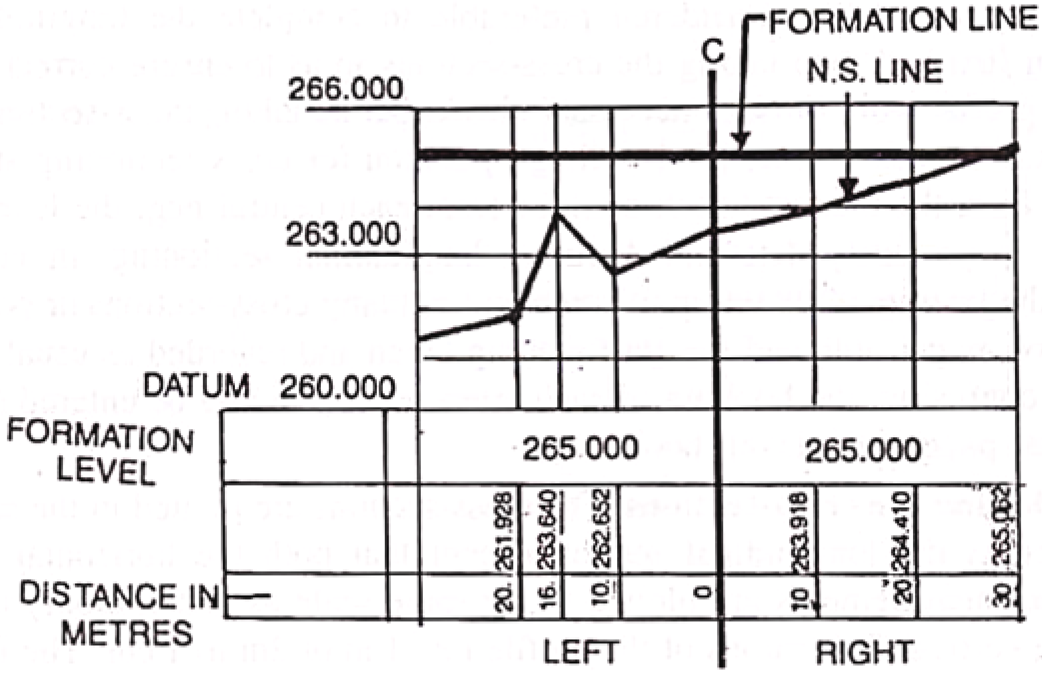


Fig. 7.34

Levelling for Giving Levels for Works:

In the execution of works, it often becomes necessary to give levels or marks up to which a work is to be constructed. This is done by driving a peg or making a mark either at a desired level or at a known distance above or below it. In the first case, knowing the R.L. of the desired point and the height of the instrument (H.I.), the staff reading required at that point is first calculated by subtracting the R.L. of the desired point from the height of the instrument (H.I.).

The staff is then lowered or raised until the desired reading is obtained and the point is marked suitably. In the second case, the staff reading of an arbitrary point is first ascertained as explained above and difference between that point and the actual point of construction is determined.

To avoid confusion, this difference may be an exact number of metres. Levels given by this method are transferred to the actual points of construction afterwards when required. This may be done by using sight rails and bonding rods or a straight edge and a spirit level.

Barometric Levelling:

It is a process of levelling based up on the fact that the atmospheric pressure varies inversely with the height. In this method, a barometer is used to measure the atmospheric pressure and thus the elevation of points. It is not an accurate method and so is chiefly used for rough type levelling works such as during reconnaissance.

Hypsometery:

This method of levelling is based upon the principle that the boiling point of water changes with the height. Hypsometer is used to find the boiling points of water and thus the height of the points. It is also an approximate method.

Trigonometrical Levelling:

It is the method of levelling in which the heights of points are calculated from the horizontal distances and vertical angles measured in the field.

§ 59. Trigonometric leveling.

Trigonometric leveling is performed when solving various engineering problems in order to create a high-altitude basis for a topographic survey, as well as when performing the topographic survey itself.

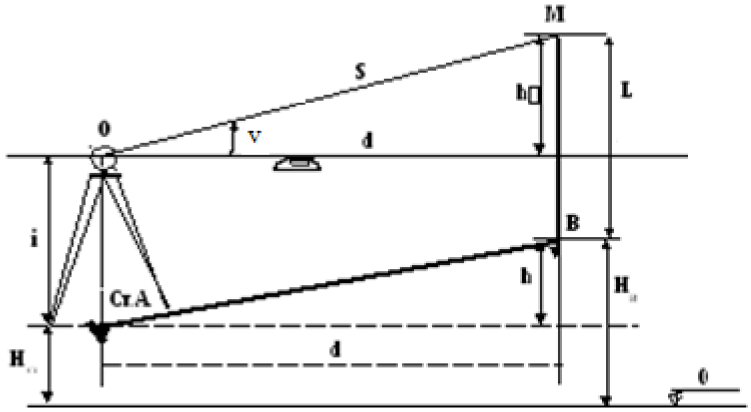


Fig.78. Trigonometric leveling.

To determine the excess *h* at point *A* , a theodolite is installed (Fig. 78), and at point *B a* milestone or rail, point the pipe to the top of the sighting mark and measure the angle of inclination *v* . The horizontal distance *d* , the height of the theodolite above the point *a* , the height of the sign *l should be measured* .

From the figure follows:

$h + l = h' + a$ or *h \ u003d h ′ + a - l* _ (189)

Since $h' = d$ tg v then $h = d$ tg $v + a - l$ (190)

Taking into account the curvature of the Earth and the influence of the refraction of the light beam

$$h = d \text{ tg } v + a - l + f, \tag{191}$$

where *f* - correction for the curvature of the Earth and refraction

$$f = \frac{0{,}42 d^2}{R} \quad . \tag{192}$$

Shorthand formula for trigonometric leveling (used when $l = d$)

$$H = d \text{ tgv} . \_ \tag{193}$$

When determining distances with a filament rangefinder

$$d = n \cdot k \cdot \cos^2 v + c . \tag{194}$$

From here

$$h = n \cdot k \cdot \sin v \cdot \cos v + c \cdot \sin v = \frac{1}{2} n \cdot k \cdot \sin 2v + c \cdot \sin v .$$

For tilt angles not exceeding 10°, $\sin \nu \approx ½ \sin 2\nu$,

Therefore $$h = \frac{1}{2}(k \cdot n + c) \cdot \sin 2\nu . \qquad (195)$$

In the case when a trigonometric leveling course is laid to determine the heights of points, the allowable residual of elevations in the course is determined from the expression

$$_{add} f_h = \pm\left(0{,}04 \cdot S \cdot \sqrt{n}\right) \text{м} \qquad (196)$$

where S is the average length of the travel line, expressed in hundreds of meters,

n is the number of lines in the course.

In order to reduce the influence of refraction, trigonometric leveling at large distances should be carried out 2–3 hours after sunrise and no later than 2–3 hours before sunset.

To speed up work on trigonometric leveling, instead of theodolites, special combined geodetic instruments can be used - total stations with automatic registration of measurement results.

barometric leveling. This type of leveling is based on the relationship between the height of a terrain point and the value of atmospheric pressure at a given point. Due to its simplicity and high productivity, barometric leveling is used when performing geological surveys in hard-to-reach forested and, mainly, in mountainous areas. However, the accuracy of the method is low - the error in determining the heights reaches one or more meters.

There are a number of formulas that establish the relationship between the difference in heights of terrain points and the measured pressure values at these points. The most common in production conditions is the following:

$$h = h_{st} \Delta B . \_ \qquad (197)$$

Here $\Delta B = B_1 - B_2$ is the difference in atmospheric pressure at the observed points; h_{st} - barometric step (the height to which you need to raise or lower the barometer so that its readings change by 1 mm Hg - about 10 m at sea level):

h_{st} \u003d 8000 (1 + αt) 1/ B , (198)

where $t = \frac{t_1 + t_2}{2}$, $B = \frac{B_1 + B_2}{2}$ are the average temperature and average pressure, respectively; $\alpha = \frac{1}{273{,}2}$. Due to the uneven distribution of air density, due, in particular, to the difference in temperatures at points lying at the same height, the pressure can be different, i.e. *isobaric surfaces* - surfaces on which the atmospheric value is the same - are not parallel to level surfaces.

Therefore, the distance between *isobars* - the lines of intersection of isobaric surfaces with the level - is not constant.

The change in pressure in the direction of the normal to the isobars over a segment of 111 km (the length of the meridian arc is 1 ° at the equator) - *baric gradient* - averages 0.01 - 0.02 mm Hg / km. The uneven state of the atmosphere also causes a temporary change in pressure at the same point, which averages 0.3 mm Hg / h, reaching ± 1.5 mm Hg / h, which can lead to a gross error in determining excess. For example, when measuring pressure at the second point one hour later than at the first, it can lead to an error in determining the elevation up to 15 m.

The determination of the average air temperature is not always reliable. In practice, it is defined as the average of measurements at several points. At the same time, temperature changes in the horizontal direction

at a distance of 10 - 20 кмcan reach 8-10 °C, and even temperature inversions are observed in the vertical direction. An error in determining the temperature of 2 °C leads to an error in determining the excess in $\frac{1}{100}$
.

To reduce the influence of these factors on the accuracy of determining the excesses, barometric leveling is carried out according to a certain method, which provides for fixing changes in pressure and temperature at strictly defined time intervals, limiting the length of the route and leveling time, performing observations at points only in clear weather, in the morning and evening hours . In barometric leveling, an aneroid barometer is usually used to measure pressure, which is previously calibrated according to the readings of a mercury barometer, as a result of which a certificate is compiled for this aneroid, in which aneroid corrections are given . The time of observations is fixed according to verified chronometers.

Currently produced microbarometers (OMB, MBNP, MB) make it possible to increase the accuracy of pressure readings on the instrument, however, this does not lead to a significant increase in the accuracy of determining excesses due to the nature of the causes of pressure fluctuations, which were mentioned above.

Each aneroid reading is corrected for a mercury barometer reading. So, for example, if A is an aneroid reading, then the actual atmospheric pressure that a mercury barometer would show at 0 ° C and normal gravity (at sea level and at a latitude of 45 °) can be found by the formula

$$B_o = A + a + bt_A + c\,(760 - A), \qquad (199)$$

where a is the difference between the readings of a mercury barometer and an aneroid at $t_A = 0$ and normal pressure; bt_A is temperature correction; b is the temperature coefficient; $c\,(760 - A)$ - scale correction, depending on the change in air pressure, c - coefficient of proportionality.

To determine the difference in the heights of any two points, it is necessary to record on each of them the readings of the aneroid A , the temperature of the aneroid t_A , the outside air temperature, the height of the aneroid (usually the aneroid is held at chest height) and indicate the time of measurement (year, month, day, hour and minutes). At each point, the aneroid is allowed to settle for 15-20 minutes and then a count is made.

Leveling with two aneroids. In the middle of the section at the station, one of the observers is located, who regularly records the readings of the aneroid (station) every 20-30 minutes. The second observer, after comparing the readings of his aneroid with the station's, goes to all the points planned for leveling and records the readings of the instruments on them. Upon completion of work and return to the station, readings are again recorded on field and stationary instruments. This is how monitoring is done. The discrepancy between the moments of field measurements and the station ones is of no importance, since for any reading from the field aneroid it is possible to calculate by interpolation the reading that would have been obtained at that moment at the station. To calculate absolute heights, the station height must be known.

Leveling with one aneroid is performed by an observer who records instrument readings at each point, returns to the starting point of the route and makes a control record of instrument readings there. The resulting discrepancy in pressure is distributed with the appropriate sign to all observed points in proportion to time. This correction is called the time correction. The values of the barometric steps are determined according to the tables (for example, Khrenov L.S. Tables for barometric leveling. - M., Nedra, 1970).

Hydrostatic leveling

This type of leveling is based on the use of the properties of liquid levels in communicating vessels. If you take two transparent vessels (glass tubes), place them in frames with graduations, connect them with a hose, fill them with liquid, then the difference in readings of the liquid levels in the vessels can determine the excess. The error in determining the excesses does not exceed 0.1 - 1 mm. The method is used for leveling the foundations of turbines, compressors, installation of plumbing equipment.

Airborne radio leveling is carried out from aircraft using a radio altimeter and a statoscope operating on the principle of a differential barometer.

A radio altimeter measures the height from the aircraft to the earth's surface, a statoscope measures changes in the aircraft's flight altitude relative to an arbitrarily chosen surface.

At the beginning of the route, the aircraft flies over point A with a known height H_A fig. 50 and measures the flight altitude with a radio altimeter Z_A. At the same time, the statoscope reading is recorded to determine h_A (a statoscope is a differential barometer that measures changes in pressure; changes in flight altitude h can be determined from a change in pressure).

Further, the pilot flies over the route points 1, 2,.., n and at each point fixes Z_n and hn . _ The heights of these points can be calculated using the formula

$$H_n \backslash u003d H_A + (Z_A - h_A) - (Z_n - h_n). \qquad (200)$$

At the end of the flight, the aircraft flies over point B , whose height H_B is also known and measures Z_B and h_B. Calculate $H_{Vism} \backslash$ u003d $H_A + (Z_A - h_A) - (Z_B - h_B)$.

Compare H_{Vizm} and N_B and find the pickup f_h which arises due to the curvature of the isobaric surface *PP* during the flight T

$$f_h \backslash u003d H_{Vism} - H_B. \qquad (201)$$

Calculate corrections to the measured point heights by the formula

$$\delta_{hn} = -\frac{f_h}{T} \cdot t_n, \qquad (202)$$

where t_n *is* the flight time from point A to point n . Corrections are made to the measured heights and get H_n corrected

$$H_{n\ correct} \backslash_{u003d} H_n + \delta_{\mathrm{hn}}. \qquad (203)$$

Errors in determining heights during airborne radio leveling are ±2.5 m.

Chapter 13. Use of modern geodetic instruments in solving geodetic problems.

§ 60. Automatic theodolites and levels.

Special electronic sensors take readings and then display the results on the monitor. These tools are indispensable in construction during the construction of various infrastructure facilities, as well as for:

automation of angular measurements;
formation of lines of geodetic points in the implementation of construction activities;
engineering and geodetic research to create topographic materials;
in the military field.

Modern manufacturers offer mostly digital models. To make sure that the characteristics correspond to the declared figures, it is recommended to periodically check the devices.

Fig.79. Digital level VEGA

Classification and types

Depending on the design, electronic theodolites can be equipped with a vertical compensator or a laser/optical type plumb bob. They are also divided into different types according to the degree of RMS (root mean square error):

technical - RMS from 15 to 30;
construction, precision - from 2 to 10;
engineering, high-precision - from 0.5 to 1.

The principle of working with any electronic theodolite is quite simple, you can use it even in the absence of special skills, after a preliminary briefing.

Principle and features of devices

The most important difference between electronic theodolites and optical devices is the use of a binary - digital measuring system with full rotary angle sensors. Its essence lies in the marking of a photoelectric disk with an algorithm of black and white code marks, when illuminated, it turns out 1 or 0. This value is subsequently analyzed and processed in the processor. The captured information is recorded in an integrated storage element or transferred to external media or a PC.

The main components of electrotheodolites are:

laser or optical plummet;
stand with tribrach;
graphical LCD screen with control panel for the most important actions;
visual optical tube with a network of threads for high-quality positioning on the object;
screws for fixing, adjusting and aiming;
high-strength case with a reference system placed in it.

Modern models, as a rule, are equipped with vertical compensators, which greatly simplifies the work with the device.

Popular Devices

The modern market offers many models of electronic theodolites. Among the most popular devices are:

- **The DJD2-GH** is an accurate instrument with a mean square error of 2. A two-line graphic monitor and control panel are located on opposite sides of the alidade. The vertical and horizontal angle data is displayed on the monitor at the same time. The 30x tube provides a straight image. Thanks to the built-in backlight, the device can be used even in poor lighting conditions.

- **ADADigiTeo 10** with a dust and waterproof case and a double-sided LCD screen, the numbers on which are visible in any position of the device. The angular accuracy of the telescope when zooming 30 times reaches 10, the picture is formed straight. The presence of a code dial minimizes inaccuracies when reading data.
- **DT-205** with detachable tribrach to make it easier to set up the instrument on travel stations. Refers to construction theodolites (angular accuracy is 5). The results of the countdown are displayed on a sectional dual monitor with backlight. Information is transmitted to external storage elements or a PC using a COM port.

Benefits of using

Fig.80. Digital automated level TOPCON

The main advantages of using electronic theodolites include:

automatic calculation and fixation of reference information;

minimization of errors in determining readings (which is typical for optical models);

Possibility of operation in the dark with a laser pointer and monitor microscale illumination.

The use of devices in the field is limited due to the impossibility of their operation without batteries, recharging. Also, the electronic elements of the device (monitor, microprocessor, sensors) do not function at low temperatures.

Samples of theodalites

Electronic theodolite VEGA TEO-5B Electronic theodolite TE-20 GEOBOX

Electronic theodolite ZIPP02 GEOMAX

Fig.81. Samples of electronic theodolites

Electronic (digital) levels

Electronic (digital) levels are modern multifunctional geodetic instruments that combine the functions of a high-precision optical level, an electronic storage device and built-in software for processing the measurements. The main distinguishing feature of electronic levels is a built-in electronic device for taking readings on a special rail with high accuracy. The use of electronic levels allows you to eliminate personal errors of the performer and speed up the measurement process. It is enough to point the device at the rail, focus the image and press the button. The device will perform the measurement, display the obtained value and the distance to the rail on the screen. Digital technologies can significantly expand the capabilities of levels and their areas of application. Experience shows that with a digital level, a 50% time saving is achieved compared to a conventional level. The main reasons are the fast data acquisition and the saving of measurements in the instrument's internal memory.

Examples of electronic levels : **TRIMBLE DINI 0.7, Sokkia SDL30M, Topcon DL-101C, Leica Sprinter 250M**

Trimble DINI 0.7 digital levels are ideal for accurate electronic measurement of distances and heights. The Trimble DiNi provides maximum performance for everyday surveying work. It has a rugged design (with IP55 dust and water resistance) to withstand harsh field conditions. Backlighting of the screen and circular level will allow you to continue working even in the twilight

SPECIFICATIONS:	
Accuracy (per 1 km of double run), mm: - invar rail with barcode markings - standard rail with barcode markings	0.71.3
Magnification of the telescope, times	26
Minimum viewing distance, m	1.3
Image	direct
Compensator operating range	fifteen
Keyboard	19-key alphanumeric with 4-way navigation key
Internal memory, data lines	up to 30 000
Display	graphic, 240 x 160, monochrome, backlit
Water and dust protection	IP55
Operating temperature range, °C	-20 to +50
Source of power	internal battery - lithium-ion, 7.4 V / 2.4 Ah
Working time, h	thirty
Weight, kg	3.5

The Sokkia SDL30M electronic level combines convenience and ease of use with ease of learning. To perform measurements, the user just needs to aim at the staff and press just one key, after which the SDL30M level will calculate the elevation and measure the distance. The measurement results are displayed on the screen and can be stored in the instrument's memory.

The SDL30M is undemanding to observing conditions and can be used in adverse conditions such as uneven lighting, convective air movement and vibration.

The SDL30M digital level uses a charge-coupled device (CCD) to read off a special bar code. Such measurements exclude the possibility of taking an incorrect reading and personal errors of the observer. To work with SDL30M, durable fiberglass rails with a special RAB bar code (BGS40, BGS50, ND 345124, BIS20, BIS30) are used. The level allows you to perform measurements not only on a barcode staff, but also on a conventional leveling staff, which greatly expands the possibilities of using the device.

Internal software: setting out marks and distances, calculating marks, laying a leveling path

SPECIFICATIONS :	
Elevation measurement accuracy (per 1 km of double run), mm: - with invar rail - with fiberglass rail	0.61.0
Magnification of the telescope, times	32
Distance measurement accuracy, mm	10 - 20 mm depending on distance
Measurement time, sec	3
Keyboard	8 keys
Measurement range, m	1.6 - 100
Compensator operating range	±15
Display	LCD graphic, 128x32
Image	direct
Memory	2000 measurements (64 kB)
Working temperature, °C	-20 to +50
Operating time from one accumulator, h	16
Weight, kg	2.4

The DL-101C/102C digital level with 0.4 mm/1.0 mm leveling errors increase the speed, accuracy and productivity of field work. They can save measurement data to internal memory or to Compact Flash memory cards. The internal memory is designed to store measurements of 8000 points.

When using DL-101C/102C levels complete with barcode rails, you can automatically determine distances and elevations immediately in digital form.

Main areas of application: leveling networks; tracking deformations (monitoring ground subsidence); engineering survey; survey (areal leveling, tracing, etc.); road construction (longitudinal and cross sections, removal of marks); construction of tunnels and mines.

SPECIFICATIONS:	
Image	direct
Magnification, times	32
Lens diameter, mm	45

Resolution, "	3.0
The smallest focal length, m	1.0
Compensator accuracy, "	±0.3
Weight, kg	2.8
Measurement of excesses, accuracy (RMS per 1 km), mm: - electronic reading - optical reading	±0.4 (on invar rail)±1.0
Reading resolution, mm	0.01/0.1
Measurement accuracy, cm	1-5
Measurement range, m: - fiberglass or aluminum rail - invar rail	2-1002-60
Measurement interval, sec	3
Display	Backlit LCD, 2 lines x 8 characters
Internal memory, measurements	for 8,000
Memory card, GB	Compact Flash up to 2
I/O ports	RS-232C
Keyboard	alphanumeric
Auto-off timer	there is
Horizontal circle, °	360
Food	6 AA batteries or 1 BT-31Q battery
Working time, h	ten
Operating temperature range, °C	-20 to +50
Water resistance	IPX6

The Leica Sprinter 250M digital level is ideal for a variety of high-precision construction jobs. To measure, just press one key and the result will immediately appear on the display.

SPECIFICATIONS:	
Distance measurement range, m	2-100
Distance measurement accuracy on a high-precision rail, mm/m	10/10
RMS per 1 km of double run, m m: - with a barcode rail - with a barcode fiberglass rail - with an engineering rail	1.00.72.5
Minimum focal length, m	0.5
Operating range of the compensator, '	ten
Memory, measurements	1000
Standard measuring programs	measurement of distances and elevations, determination of height difference, monitoring function, leveling, determination of filling / excavation, saving and transferring data

Keyboard	5 buttons to control the interface, measurement button
Magnification of the telescope, x	24
Display	LCD, 128x104
Image	direct
Field of view, °	2
Lens diameter, mm	36
Round level sensitivity, '/mm	10/2
Round level mirror	there is
Compensator type	magnetic damper
Operating temperature range, °C	-10 to +50
Dust and moisture protection	IP55
Device dimensions (LxWxH), mm	219x196x178
Weight, kg	2.55

§ 61. Principles of operation of automatic geodetic instruments .

Electronic total stations can be divided into three groups - the simplest, universal and robotic.

The simplest are electronic total stations with minimal automation and limited built-in software functions. The error in their measurements of horizontal and vertical directions is 5-10", distances - 5-10 mm per 1 km.

The electronic memory of total stations allows you to store in digital form information about the position of 500-1000 points, which can be recorded on a removable memory card.

total station (Fig. 3.5) can be used to create a boundary survey network, to determine flat rectangular coordinates of boundary marks and characteristic points of a property. The composition of the tacheometer includes: an electronic theodolite, a light distance meter, a computing device and an information recorder, a control panel (controller) and a display on which alphabetic identifiers and digital information are displayed.

The total station set includes: a reflector, stands, power supplies, milestones, tripods, discharging devices and other accessories. Electronic total station ZTa5 has the following measurement accuracy characteristics, characterized by root mean square errors: horizontal angle 5", vertical angle 7", slope distance *D* from 2 to 2000 m - (5 mm + 3D10 $^{-6}$) mm.

Fig.82. Electronic total station ZTA5 **:**

1 - display; *2* - sighting tube; *3* - power supply; *4* - keyboard

Universal electronic tacheometers include a large number of built-in programs that allow solving engineering land management and cadastral tasks in the field. The measurement error of horizontal and vertical directions by total stations of this group is 1-5", distances - 2-3 mm per 1 km. The electronic memory of total stations can store in digital form information about the position of up to 1900 coordinates of the measured points.

The main component of the electronic total stations of both groups is the controller module, which is a field computer and a total station control panel. The functionality of the total station depends on the controller, such as performance, memory size, screen type, presence and number of built-in programs. Most electronic total stations have a built-in controller controlled from a numeric or alphanumeric keypad. The number of keyboard keys depends on the type of total station and the number of tasks performed by the controller. Recently, field graphic computers with an active screen have been used as controllers, which allows using an electronic pencil to control the operation of the total station, the measurement process, and also to view a graphical display of the results of work in real time.

The unique "Total Station" system (complete station) includes an electronic total station, one of the modules of which is a single-frequency satellite receiver installed in place of an additional keyboard, and an antenna fixed on top of the transport handle.

Robotic electronic total stations include a servo drive, which, when used, increases the measurement performance by about 30% and reduces the likelihood of gross errors in measurements when aiming at sighting targets. Servo commands are generated by special electronic tracking devices.

The electronic total station Geodimeter 600 (Sweden) has a four-speed servomotor in its design, which provides pointing at the reflector in the search and tracking modes. The active reflector included in the tacheometer set is an active emitter (light-emitting diode), the radiation of which is recorded by the automatic guidance and tracking system located in the tacheometer telescope.

Fig.83. Electronic total station Geodimeter 600

Distances are measured with an electronic total station using an electromagnetic range finder (light range finder) built into it, the principle of operation of which is based on the phase method of measuring distances.

The laser scanner is a geodetic instrument that automatically takes measurements of points in a given sector at a given interval, and includes: a high-speed laser rangefinder and a laser beam redirection system that helps determine the scanning area. The principle of operation of a laser scanner is based on identifying a sufficient number of points (three-dimensional coordinates along the *X*, *Y*, *Z axes).* The object is measured using a range finder.

Leica ScanStation 2 **RIEGL Air Scanner LMSQ 780**

Fig.84. RiCOPTER unmanned system equipped with RIEGL air scanner

The rangefinder makes up to several thousand measurements per second, storing and sending data on each measurement to the computer memory. The laser beam is deflected vertically by a stepper motor with a mirror, and horizontally the deflection occurs when the scanner itself rotates. As a result of such actions, it becomes possible to obtain three-dimensional coordinates of each point, forming groups called "point clouds". Some laser scanners are equipped with a digital camera that allows you to take panoramic pictures of the subject's surroundings.

All measurement data, as well as images, enter the memory of a portable computer, the data and the surface of the scanned part are stored, analyzed and displayed on the screen in the form of a three-dimensional image. Measurements are carried out with high density and accuracy, which allows creating a three-dimensional mathematical model of the subject. The scanner automatically takes measurements at distances up to 300 m at a speed of up to 50,000 points per second, while maintaining high measurement accuracy - an error of up to 6 mm at 50 m. High resolution (1 mm at 300 m) and a small spot of the laser beam (4 mm at 50 m) allow you to perform high-quality field measurements.

Laser scanners can be contact and non-contact. The latter are the most convenient to use, since no contact of the equipment with the surface is required to obtain high-precision data when surveying.

Non-contact scanners are divided into two types: active - they direct laser beams at an object, after which they analyze the data obtained by reflection; passive - output data about the model using the analysis of ambient radiation, but the illumination of the scanned object must be accurately and perfectly matched.

Currently, such companies as Leica Geosystems (Switzerland), Trimble (USA), Zoller+Frohlich (Germany), Faro Technologies (USA), Riegl (Austria) and others are developing devices for laser scanning.

Digital (*electronic) levels* are currently used in land cadastral work, complete with a special barcode rail, which allows you to automate the reading. Electronic levels are modern multifunctional geodetic instruments that combine the functions of a high-precision optical level, an electronic memory device and built-in software for processing measurement results. The use of electronic levels allows you to eliminate the personal errors of the performer and speed up the measurement process.

Laser levels are electronic-mechanical devices in which the principle of laser beam rotation is applied. The luminous flux emitted by the built-in LED is focused using a lens or prism, due to which the level allows you to get a laser dot or line on the surrounding objects. Therefore, laser levels are divided into the following categories:

positional (prism), using one or more LEDs that create a narrow light flux, which is converted into a laser plane with the help of a prism. Laser levels, equipped with two LEDs and two prisms, project two laser planes - vertical and horizontal. Intersecting at right angles, they form a cross. Using the keys on the housing, the user can leave only one of the laser lines on. The sweep angle of the laser beam around the positional level is 120°. Existing designs of multiprism levels make it possible to obtain three or more laser planes, and some models emit points on surrounding objects;

rotational, allowing to obtain a laser plane due to the rotation of the LED by the built-in electric motor. Rotary levels can project the laser plane in a 360° plane.

Global navigation satellite systems (GNSS) have significant advantages over traditional geodetic systems:

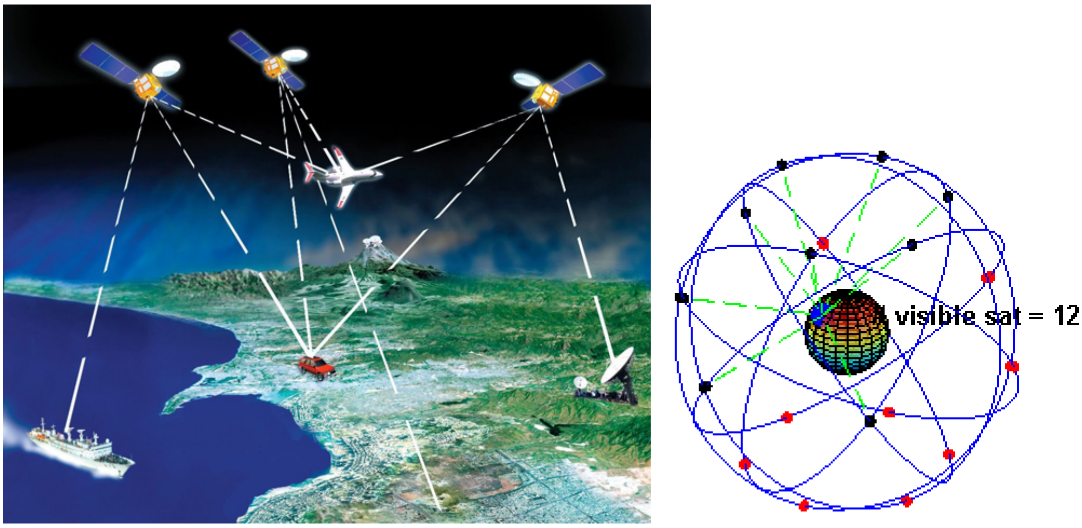


Fig.85. Scheme of GNSS operation

the need to locate defined points of geodetic networks, for example, reference boundary lines, is eliminated, provided that they are mutually visible and the distance between the defined points can be tens of kilometers; observations are possible in any weather, both in the daytime and at night;

measurements and processing of results are almost completely automated;

it is possible to obtain the coordinates of geodetic points, turning points of the boundaries of land plots, survey stations, characteristic points of real estate objects in real time; reducing the time spent on shooting, and improving the accuracy of the results.

Currently, there are two global navigation satellite systems - GLONASS (Russia) and GPS (USA). They include three main segments: the control and management segment is a complex of ground-based facilities that provide continuous monitoring and control over the operation of the entire system. One of the components of this segment is a space geodetic network evenly located on the surface of the Earth (Fig. 85.);

space segment - a constellation of navigational artificial Earth satellites revolving around the Earth in certain orbits. The dimensions and shape of an elliptical orbit are determined by the size of its major semiaxis *a* and eccentricity *e;* consumer segment - receiving equipment of the consumer segment (GPS receivers), which solves a complex of tasks such as receiving and primary processing of signals from satellites.

Signal processing is performed in order to generate the information necessary for consumers - space-time coordinates, direction and speed, spatial orientation.

The receiving device (GPS-receiver) performs the primary signal processing. The signals are sent to the search and measurement unit of the GPS receiver. After the search is completed, the signal is captured, which enters the computing unit. At the operator's instruction, the results of signal processing are reflected on the display.

There are three classes of GPS receivers: the first one is intended for fast navigation determinations of coordinates. This type of receivers is convenient to use for reconnaissance, stakeout and shooting of objects with low accuracy;

the second, designed to determine the position of moving objects;

the third, which, as a rule, refer to receivers for geodetic purposes. They have a multi-channel unit that simultaneously monitors the signals of several satellites (up to 12 or more). The internal memory of the receiver is up to 100 MB or more. The receivers are equipped with ports for integration with other equipment, including computers.

Structurally, GPS receivers are made in the form of separate or combined units that contain: an antenna device, a controller (computer with a keyboard) and batteries. With the help of a controller, built-in or attached to the receiver, the user can manage and control the process of satellite observations. Often, receivers have a built-in radio modem, with which you can transmit or receive the necessary information via communication channels in real time.

The choice of a specific class of satellite signal receiver for carrying out land cadastral geodetic works depends on the required accuracy of determining the position of objects.

The personal GPS navigator receives signals from GPS satellites and calculates the geodetic coordinates (latitude, longitude, and altitude) of your current location. In addition, with the help of GPS-navigators, you can remember the received coordinates in the form of waypoints and make routes from them. Guided by the waypoint, they determine the direction to the desired point and automatically memorize the coordinates in the direction of travel (travel log). These devices have the ability to download and display an electronic map in their own format. Some models of GPS navigators have a built-in magnetic compass and barometric altimeter. GPS navigators use signals transmitted from satellites to determine their location. Signals contain two types of information - navigational messages and a pseudo-random code. With their help, the absolute position of a point on the earth's surface is set in the WGS-84 coordinate system. The receivers have the option of converting the geodetic coordinates of points into flat rectangular geodetic coordinates of the corresponding map projection.

Recently, personal **GPS** navigators are becoming more widespread, providing clarification of location and integrated with a receiver of signals from wide-range differential subsystems. Such systems are the American **WAAS, the** European **EGNOS,** which consist of geostationary spacecraft and ground stations designed to collect and process satellite information.

Laser roulettes (rangefinders) are small and easy to operate. To measure distances in the roulette case there is a laser electromagnetic rangefinder. When measuring, the laser beam is directed along the laser spot or with the help of an optical sight fixed on the body of the tape measure to the reflective surface of the object to which the distance is measured.

Fig.86. Laser roulette Leica DISTO D2 NEW

To measure lines located in a horizontal plane, a built-in cylindrical level is provided in the laser tape measure, the axis of which must be parallel to the laser beam.

On the working panel of the roulette there is a backlit screen, which ensures operation in poor lighting conditions. The alphanumeric pad has control keys. The measurement results are stored in the electronic memory of the roulette.

The action of laser tape measures is in the range from 0.2 to 200 m. The distance measurement accuracy is characterized by an RMS error of 1–3 mm, without taking into account errors in the installation of a tape measure at the beginning of the measured line, as well as external conditions.

The tape measure software implements functions such as calculating and highlighting the maximum or minimum distances in the continuous measurement mode. Highlighting the minimum value of the measured distance allows you to use a tape measure when shooting a situation using the perpendicular method.

Laser roulettes have a built-in COM port, which makes it possible to export measurement results to the computer's memory.

§ 62. Automatic system of theodolite survey.

An electronic total station is a universal optical-electronic geodetic device that allows specialists to perform almost all types of work in modern geodesy with a fairly high measurement accuracy. At the same time, it can be used as a theodolite, and as a level, and as a light range finder. The versatility of this device lies in its versatility. With it, you can perform direct and indirect measurements, which are immediately displayed. These include:

distance measurements (lengths and horizontal distances);

definition of angles (horizontal and vertical);

finding plan and height coordinates.

In addition to these standard functions, an electronic total station is able to solve certain applied problems using its technical capabilities and mathematical algorithms embedded in the electronic computing part of the device. After selecting the necessary options, entering the initial data and taking measurements, after a few moments, the desired data is displayed on the screen of total stations:

coordinates of the station point of the tacheometer, etc. and the solution of the reverse geodetic resection on the ground;

slope length, horizontal distance, elevation between points, when performing the function to determine the inaccessible distance and height;

area bounded by lines passing through points with obtained coordinates after field measurements in this option;

coordinates of the theodolite traverse with linear, angular, relative, coordinate discrepancies, when adjusting this traverse and obtaining the true coordinates of points.

In addition to all this, electronic tacheometers allow you to use their functional abilities for stakeout and survey work, in the modes provided by the design:

removal of points in nature;

removal of the line on the ground;

removal of a circular line;

point projection;

offset measurements;

topographic survey ;

track shooting;

survey of the cross-sections of the route.

For the successful use of electronic total stations in the work, it is best to use the entire complex of automation of the geodetic process, using a personal computer and data transfer software. This process simplifies the analytical preparation of raw data while preventing human error. Helps with the processing of data from field surveys and stakeouts . The speed of processing the results obtained and the productivity of the entire geodetic production are increasing.

Considering the device of an electronic total station, three components should be noted in it:

optical;

mechanical;

electronic.

Optical, mechanical and even electronic parts of the device are known from opto-mechanical and opto-electronic theodolites, which are only improved by manufacturers over time.

A distinctive feature of electronic total stations is the presence of two important nodes:

rangefinder with infrared LED phase and pulse method of measuring distances and transmitting them to the liquid crystal display;

electronic digital computing device with software, various modes of operation and a panel with a display that allows you to display all the results on your screen.

As part of such electronic devices, four systems interacting with each other should be noted:

orientation;

guidance;

measurements;

management and organization of all geodetic processes of measurement, calculation and even simple adjustment;

The orientation system includes the geometry of the axes of interconnected elements, mechanical units, levels (horizontal, round, electronic), plumb fixtures, compensators and attachment mechanisms.

The guidance system includes a spotting scope with a movable optical system inside it and attachment and guidance mechanisms.

The measuring system can include devices of horizontal and vertical circles with a system of reading by limbs and digital conversion of angular values, a light-range device with mechanisms for measuring and calculating linear quantities.

The control system includes a working panel with a screen display, electronic computing and software that allows you to select the necessary modes of tasks and manage them.

Fig.87. Appearance of the electronic tacheometer.

From different sides of the appearance of the electronic total station of the Japanese company SOKKIA brand SET530RK3, shown in the image, you can see all the details and assemblies of this type of instrument. They include:

- fixing screw of the horizontal circle (1);
- micrometer horizontal guidance screw (2);
- fixing screw of the vertical circle (3);
- micrometer vertical guidance screw (4);
- keyboard panel for data entry in numerical and alphabetic form (5);
- display screen, for visual output of all data (6);
- ampoule of a cylindrical level for leveling the device (7);
- correction screws for adjusting the cylindrical level (8);
- eyepiece (9);
- eyepiece focusing (10);
- focusing telescope (11);

- sighting device (12);
- pulse LED indicator (13);
- screws for fixing the upper handle (14);
- a handle used to carry the tool (15);
- place for fixing the compass (16);
- battery compartment latch (17);
- battery compartment (18);
- total station stand (19);
- lifting screws to bring the device into working position (20);
- connector for connecting external power devices (21);
- connector for connecting a cable for transferring files (22);
- round level to bring the tool axis into a vertical state (23);
- corrective screws for adjusting the round level and bringing it into working condition (24);
- tool stand base plate (25);
- stand fixing latch (26);
- focusing of the thread plummet of the optical plummet (27);
- optical plummet eyepiece (28);
- point corresponding to the height of the tool (29);
- place of infrared radiation (31);
- lens (30);
- plummet point (32).

In geodesy, high-precision work requires the use of techniques with measurements in the positions of the telescope with the circle right (KP) and the circle left (CL). For convenience in the geodetic production process, it is necessary to have control panels on both sides of the total station.

Technical characteristics of total stations

Regardless of the manufacturer, all electronic total stations have one range of technical characteristics that have certain qualitative differences. The main ones that are necessary for choosing the appropriate tool are:

- sizes and magnification of telescopes can be 26, 30, 36, 40 times;
- type of image, structurally usually laid down is a direct image;
- distance measurement ranges: on a prism up to 6000 m, on a film up to 800 m, in reflectorless mode up to 350 m
- angular root-mean-square errors having values of 2, 3, 5, 6 seconds;
- automatic tilt angle compensator with a compensation range of three to six minutes, which is a liquid two-axis sensor;
- linear root-mean-square errors depending on measurement modes:
- exact (single, multiple, averaged);
- fast (single or multiple);
- when measuring on a prism, linear errors (LFR) are within ± 2mm for accurate and ± 6mm for fast measurements;
- when measuring on film, linear SCPs have values for accurate ± 3 mm, for fast ± 6 mm;

- in reflectorless mode, the values of the SCP fluctuate depending on the range of devices capable of operating in this mode. They can range from ± 3mm to ± 15mm;
- power sources are usually lithium-ion batteries;
- pulse sources are red spectrum LEDs of the second, third class;
- tool centering is achieved with an accuracy of 1 mm, using an electronic level in the range of no more than three minutes at a height of 1.3 m; **Accessories**

  To achieve all the technical characteristics when measuring with electronic tacheometers, auxiliary equipment is used along with them. It is important to note that it is advisable to select all additional devices complete with the main device of the same manufacturer. You can give a whole list of such accessories, which include:

- a portable personal computer (laptop) for automating the entire process of geodetic field and office work;
- tripods, tripods with wide heads for ease of installation and fastening of the total station, heavy in weight and made of wood or polymers (fiberglass);
- corded plumb line, designed to set the tripod over the point and precise centering of the device;
- compass, for orienting the instrument on the ground towards the northern direction;
- diagonal nozzles (mounted on the eyepiece) used for the convenience of observations, aiming at significant tilt angles (up to 90º) of the telescope;
- different sun filters;
- cable and storage devices (flash memory) for data transmission;
- prisms (miniprisms), for receiving and reflecting signals;
- prism holders;
- reflectors and reflective films;
- reflector height adjustment adapters;
- adapter adapters for external and internal fastening of reflectors;
- milestones for the visibility of reflectors;
- tripods, bipods for installing a milestone with a reflector;
- rechargeable batteries and chargers with it.

Verification of electronic total stations

In addition to standard verifications of geodetic goniometric instruments, it is necessary to highlight in the first two paragraphs of the list and characteristic verifications of total stations :

- laser plummet;
- to determine the constant correction of the range finder;
- plummet axis of the optical plummet;
- perpendicularity of the horizontal axis and the grid of threads;
- horizontal position of the reticle line;
- by definition of collimation error;
- by determining the zero point of the compensator;
- plumbness of the axis of the round level;
- operating condition of the cylindrical level /

§ 63. Automatic leveling system.

Currently, automatic optical levels are widely used - devices that have a special structural unit called a compensator. The compensator serves to automatically maintain the optical axis of the level in a horizontal position. This approach significantly increases the reliability of the results obtained, facilitates the work of performers and saves working time.

The development of modern technologies has led to the creation of digital levels (photo 1). A digital level is a computer that itself performs several important functions, and also interacts with external software. Digital levels are used with special barcode rails, which can be used to measure not only elevations, but also the distance between them, i.e. continuously monitor the inequality of the shoulders. It is enough for the observer to point the device at the rail, focus the image and press the button.

Fig.92. Appearance of digital giveliers

After that, the device will automatically take a reading, highlighting it on the screen. The main difference between digital levels and optical levels is their stability. The accuracy of such devices declared by the manufacturers is 0.3-0.4 mm per 1 km of double stroke. Digital levels not only increase the accuracy and speed of work, but also eliminate one of the main leveling errors - the error of the observer.

Modern geodetic instruments, like electronic levels, have become a daily routine. Production is saturated with these devices, but so far the manufacturers do not have an unambiguous attitude towards their use. On the one hand, they believe that electronic levels are so good and perfect that there is no need to use traditional methods of work - everything will work out by itself.

In addition, advertising claims to almost double the productivity of work and the possibility of using the labor of less qualified specialists. On the other hand, there are quite a few enterprises that have stored purchased electronic levels in their warehouses for more than a year: contractors are afraid to use them, because due to the lack of a regulatory framework, difficulties arise with the delivery of field work results. Restrictions on the length of the sighting beam do not suit surveyors either: indeed, with a sighting beam length of more than 40 m, the measurement accuracy drops sharply.

The creation of high-precision leveling networks has always been considered as a complex professional task. State leveling networks of classes I and II are the main high-rise foundation of Russia. Leveling networks of classes I and II are also used to study the figure of the Earth and its external gravitational field; determination of differences in heights and inclinations of the average level surface of the seas and oceans washing the territory of Russia; deformation observations.

The manuals for most levels indicate that the desired high accuracy can be achieved when performing work by laying a double stroke, but it does not explain what is meant by this. The stated statement led to the need to create a special methodology for performing work, its theoretical and experimental substantiation.

The use of digital leveling can be considered on the example of one of the cycles of observation of deformations at the Sheksnin site of the Upper Volga HPP cascade. The structures of the Sheksninsky alignment include: the building of the hydroelectric power station, the earthen channel dam, the left and right-bank earthen dams. All of these structures have wall and soil deformation marks, as well as initial soil benchmarks. The layout of the initial benchmarks and deformation marks consists of leveling polygons of class I and II. Within each polygon, the total number of stations is plotted; as well as the obtained and allowable discrepancies calculated, according to the requirements, when performing high-precision leveling work in the process of observing the settlements of individual structures and their complexes. Accuracy was evaluated before and after adjustment. Before equalization, the value of μ1 for class I was 0.03 mm, and for class II - 0.06 mm. After adjustment and obtaining corrections for excesses, an accuracy assessment was performed, according to the results of which μ2 was: for class I - 0.04 mm, for class II - 0.11 mm.

11.LABORATORY WORKS

LABORATORY ASSIGNMENT #1
SOLVING PROBLEMS ON THE TOPOGRAPHIC MAP

To perform this work, the student must become familiar with the topographic map in detail.

On the selected topographic map must perform four tasks.

Two points A and B are selected on the topographic map at a distance of 8-12 cm from each other (the distance is taken by eye).

All calculations are made accurately, the linear measurements on the topographic map are carried out with maximum graphic accuracy (0.1 mm, expressed in map scale), angular measurements are carried out with an error of 15.

All work is carried out on a computer. Graphs and profile are made by means of a graphic editor on a computer or by drawing in pencil on paper.

Execute the laboratory work in accordance with the following procedure

Task 1 Measurements on topographical map

Measure on the topographic map the distance between point A and point B with maximum graphic accuracy:

1.1. Using a millimeter ruler and a numerical scale
(Fig.4).

1.2 With the help of a circular gauge and a linear scale (Fig.4). Draw the linear scale and mark the measured distance on it (Fig.5).

1.3 Using the caliper and the cross scale (Fig.6). Draw the cross scale and mark on it the measured distance (Fig.7). 1.4.

1.4 Calculate the relative error of the measurement and compare it with the permissible value:

$$\delta_1 = \frac{S_1 - S_2}{S_3} = \frac{\Delta S_1}{S_3} = \frac{\Delta S_1/\Delta S_1}{\Delta S_3/\Delta S_1} = \frac{1}{\underline{N}} \leq \frac{1}{300}; \qquad (2)$$

$$\delta_2 = \frac{S_1 - S_2}{S_3} = \frac{\Delta S_1}{S_3} = \frac{\Delta S_2/\Delta S_2}{\Delta S_3/\Delta S_2} = \frac{1}{\underline{N}} \leq \frac{1}{300}. \qquad (3)$$

5

The denominator must be rounded to integers with 2-3 significant digits.

Task 2: Determine Locations on a Map

2.1 Locate on a topographic map the geographic

on a topographic map determine geographic coordinates: north latitude (f) and east longitude (l), for points A and B, accurate to 1″:

north latitude:

fA=_____º_____□_____□; fB=_____º_____□_____□;

east longitude:

lA=_____º_____□_____□; lB=_____º_____□_____□.

2.2 Determine the rectangular x and y coordinates of points A and B from a topographic map to within 1 m:

xA = _______ m; xB = _______ m;

yA = _______ m; yB = _______ m.

2.3. Map points E and F with rectangular coordinates

on the map at

xE = 6057000 + 105 · N; (4) xF = 6057000 + 55 · N; (5)

yE = 6420000 + 105 · N; (6) yF = 6420000 + 55 · N, (7)

where N is the student's serial number in the journal.

Task 3. Determining angles of orientation on the map

3.1 Measure the directional angle of AB line with protractor and ruler.

to within 15′:

aAB =_____º_____.

3.2 Calculate the magnetic and true azimuths of the AB line using the measured directional angle and the orienteering diagram on the map.

3.3 Draw a diagram of orientation showing the orienting angles.

Task 4: terrain forms, elevations, slopes and embedding angles

4.1 Determine the heights of points A and B.

4.2 Determine the heights of points located at:

top of the hill - H1 , in the excavation - H2 , on the ridge - H3 , in the saddle - H4 .

4.3 Determine the slope angles along the AB line using the embedment diagram on the map.

4.4. make a profile of the terrain along the AB line at the following scales: horizontal at the map scale; vertical at 1:200.

4.5. on the map, use the scale of embedding, mark the shortest distance between points A and B, the slope of which at all sites would not exceed the value of 1º30′.

Methodical instructions

Task 1.

1.1 Determining the distance between points A and B using a numerical scale is performed in the following sequence:

Measure the distance between points A and B with a millimeter ruler and, using a numerical scale, determine the true distance between the points.

For example: AB = 69 mm; the map scale is 1:10,000;

= 69 · 10 000 = 690 000 mm = 690 m = 0.69 km.

1.2 Determination of the distance between points A and B using a compass-measurer and a linear scale (Fig.4,5) is carried out in the following sequence:

(a) The distance AB is less than 400 m.

The leg of the compass is set to the value "0" of the linear scale. The second leg is placed to the left of "0" on the millimeter scale. The value of the distance is determined as a reading on the linear scale;

b) distance AB is more than 800 m but less than 1 km.

The leg of the compass is set to the readout 800 m. The distance

= 800 m + , where determined similarly to point (a);

c) the distance is considerably greater than 800 m.

In this case a line is drawn between points A and B. The circular ratio is set equal to 800 m. By successive movement of the legs of the circular along the line AB, set off an integer number of times "n" the base of the scale.

The remaining part, not divisible by 800m, is measured similarly to point (a).

Distance AB equals = n · 800 m + .

1.3 Similarly to linear scale, using numerical scale, the distance is determined (Fig.4,6,7).

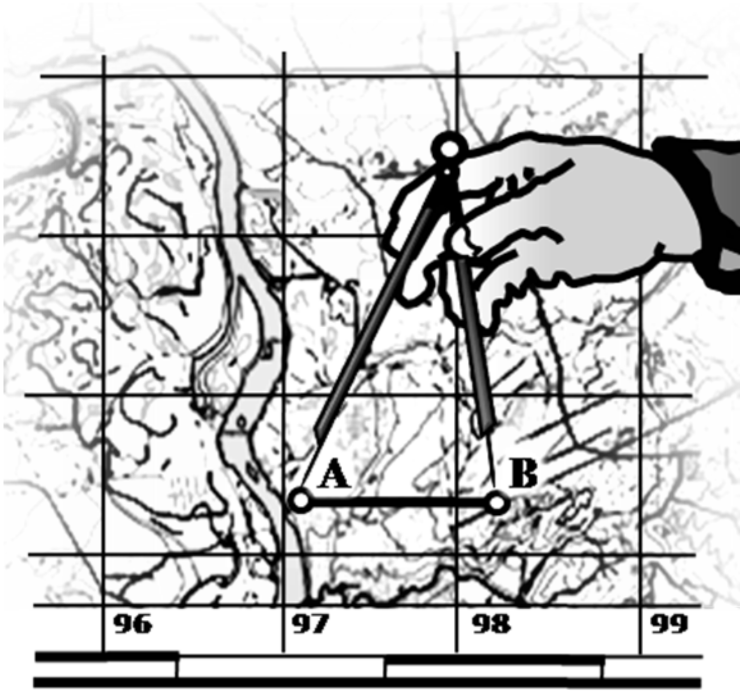


Fig. 4. Setting the circular gauge solution on the map

Fig. 5. Counting on the linear scale

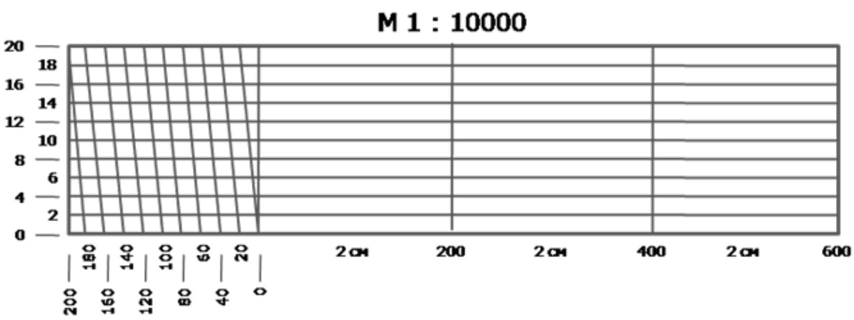
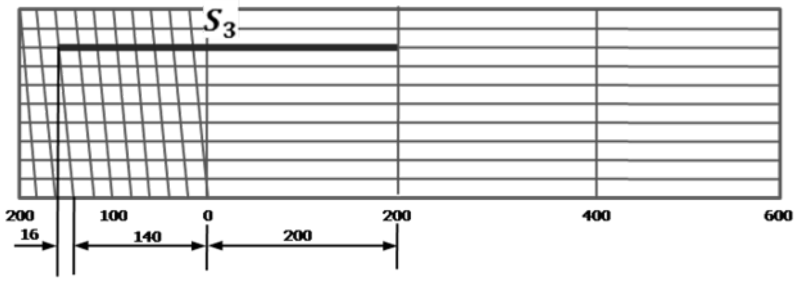


Fig. 6. Digitization of the cross scale



$S_3 = 200 + 140 + 16 = 356$ (м)

Figure 7. Counting from the cross scale diagram

Task 2.

2.1 To determine the geographic coordinates relative to the frame

Draw the western meridian 31º45′00″ nearest to point A and the southern parallel 54º37′30 (Fig. 8). The north and south lines of the inner frame of the map sheet are parallels, and the west and east meridians (Fig. 8). The map shows the coordinates of the corners of the latitude frame: 54º37′30″ and 54º40′; and the longitude: 31º45′00″ and 31º48′45″.

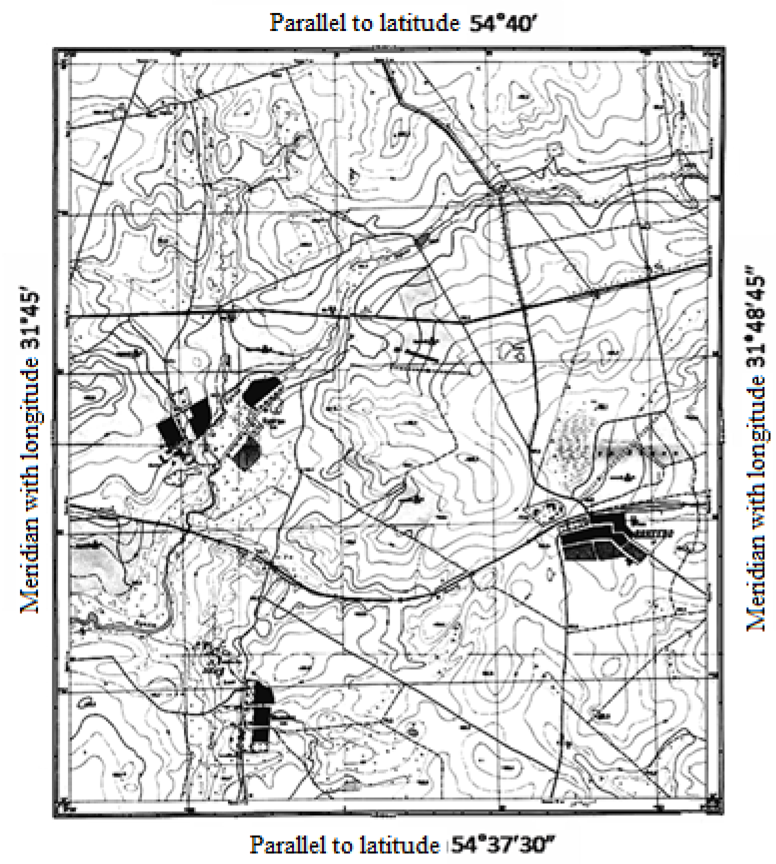


Fig. 8. Dimensions of the trapezoid of the map at a scale of 1:10000 at latitude 2º30 and longitude 3º45.

Each side of the frame is divided into whole minutes by latitude and longitude (minute frame), for ease of use the segments are filled in black. The minute frame is divided into 6 parts by dots after 10 seconds.

The latitude and longitude of point A can be obtained from the expressions:

fA=fu+Dfu ; (8)

lA =l3+Dl3. (9)

The values of and (in seconds) are found from the ratio of their segments in millimeters on the map to the segments equal in millimeters to 60 latitude or longitude multiplied by 60:

Dfu = (84 mm/183 mm) · 60″ 28″; (10)

Dl3 =(14 mm/106 mm) · 60″ 8″. (11)

The latitude f and longitude l of point A can be determined by dropping perpendiculars on the horizontal and vertical map frames. The vertical degree scale is used to determine latitude f by eye, and the horizontal degree scale is used to determine longitude l .

Thus, for point A (Fig. 9) geographical coordinates will be:

fA = 54º37′30″ + 0º00′28″ = 54º37′58″;

lA = 31º45′00″ + 0º00′08‴ = 31º45′08″.

2.2 The map has a kilometer grid of rectangular coordinates digitized from the inside of the frame. To determine rectangular coordinates through the given point B (Fig. 9) draw lines parallel to the lines of the coordinate grid and obtain the coordinate increments Dx and Dy .

First, determine the coordinate of point B relative to the nearest southwest point of the intersection of the kilometer grid lines, between which it is located.

The coordinates of point B will be equal:

XB=X0+Dx ; (12)

B=Y0+Dy, (13)

where XB , YB are coordinates of the nearest southwest point of the coordinate grid.

The values of Dx and Dy are determined by measuring the length of segments S1 and S2 on the map, and using the scale, calculate their values in meters.

For example, on a scale of 1:10 000:

S1 = 48.1 mm, S2 = 35.8 mm;

Dx = 48.1 mm x 10,000 = 481000 mm = 481 m;

Dy = 35.8 mm x 10,000 = 358,000 mm = 358 m.

Rectangular coordinates of point B:

XB = 6057000 m + 481 m = 6057481 m;

YB = 6420000 m + 358 m = 6420358 m.

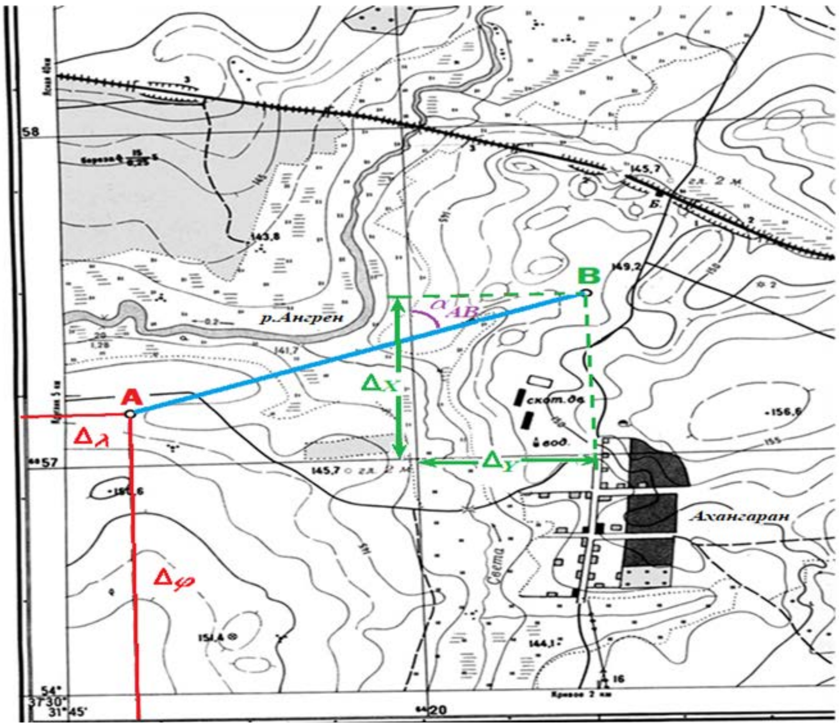


Fig. 9. Determining the coordinates of points A and B and the directive angle of line AB on the map

2.3 The mapping of points E and F with the given rectangular coordinates is done in reverse order. The coordinates of the southwest corner of the square where the point should be located are found using the given coordinate values. The resulting differences between the coordinates of that corner of the square and the point coordinates will be the coordinate increments Dx and Dy. These coordinate gradients are plotted on the corresponding coordinate axes with allowance for the map scale and the point is plotted.

Task 3.

3.1 The measurement of the directional angle is done with a protractor on the map (Fig. 9, 11).
(Fig. 9, 11).

The central mark of the protractor is fixed at any of the points
on the AB line and orient "0º" on the protractor scale in the north direction of the X-axis - a line parallel to the axis meridian (kilometer grid), count off the horizontal angle to the AB line direction clockwise.

3.2 The true and magnetic azimuth of the AB direction are determined using the orientation chart (Fig. 10) placed on the map.

For example,
a AB = 80º.

The true azimuth:
AAV = - 0º58′ = 79º02′. (14)

Magnetic azimuth:
AMAV = - 6º58′ = 73º02′ (15)
or

AMAV = AAV - 6º00′ = 73º02′. (16)

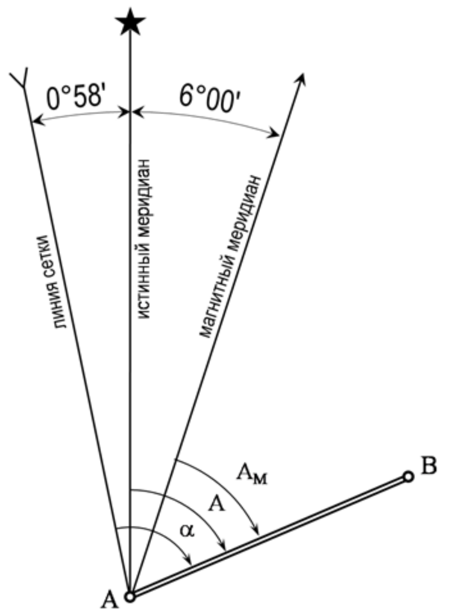


Fig. 10. Scheme of orientation

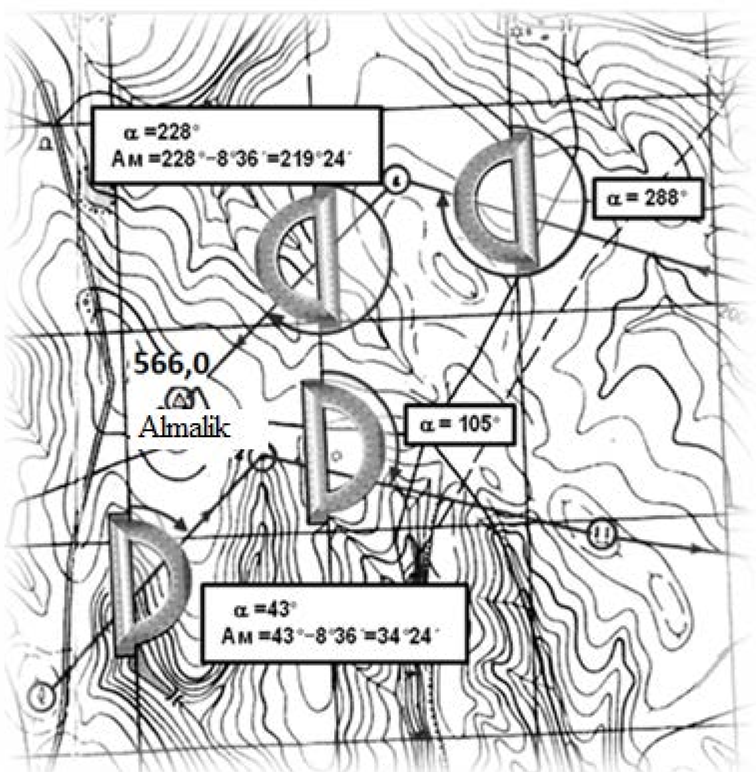


Fig.11. Measuring directional angles (a)

Task 4.

4.1 The altitude marks of the points are found using the known values of heights of the nearest contours on the map.

Lines passing through the same height points of the terrain,
are called horizontals. The difference in heights of two adjacent contour lines is called the cross-section height h (Fig. 12).

Absolute altitude is the height of a point on the terrain above sea level (Baltic).

To determine the elevation of a point, draw the shortest straight line between two adjacent horizontals (Fig. 13).

The ratio of the segments d1 and d2 is found:

d1 : d2 = 3 : 2.

Consequently (knowing the height of the section h = 2.5 m), the height of point C will be:

$$HC = H155 - \Delta h = 155 - 1.5 = 153.5 \text{ m}. \quad (17)$$

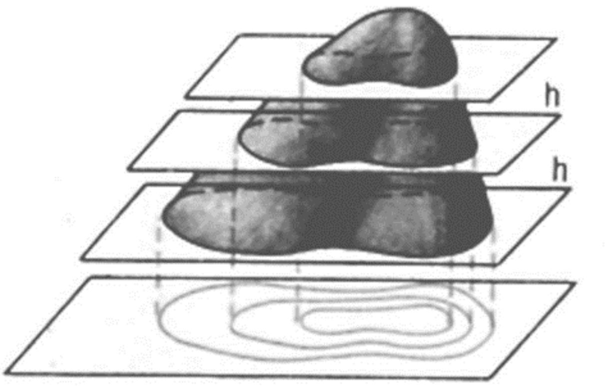


Fig. 12. Scheme of formation of contour lines

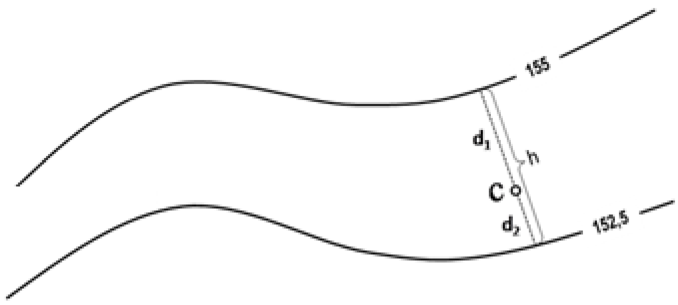


Fig. 13. Determining the height of a point by contour lines

4.2. The heights of the points located on the top of the hill H1, in the basin H2, on the ridge H3, and on the saddle H4 are determined similarly (Fig. 14).

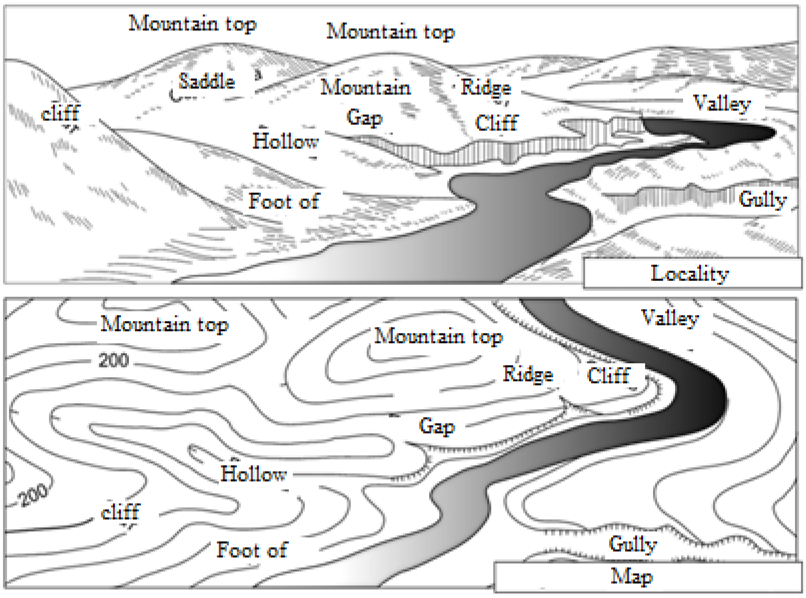


Fig.14 The heights of the points located

4.3 To determine the slope steepness (SS) (Fig. 15), characterized by the slope angle of the line of the largest slope at a given point, use the slope diagram located in the lower right corner of the map.

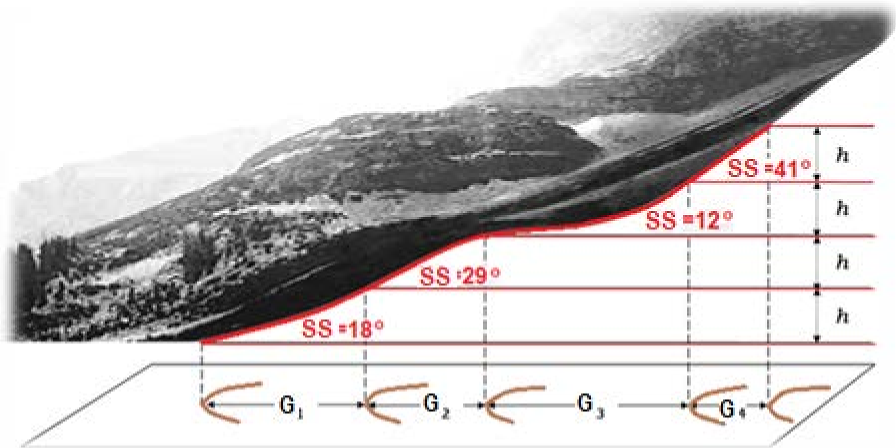


Fig. 15. Slope of the slope (SS), embedding (H)

From the map, take the desired embedment (the distance on the map between two adjacent horizontals) into the solution of the compass-measurer and transfer it to the embedment chart. Move one leg of the compass-measurer along the slope angle straight line until the tip of the second leg coincides with the curve of the embedment chart along the perpendicular to the horizontal line. The angle of inclination is read off from the first leg by eye (Fig. 16)

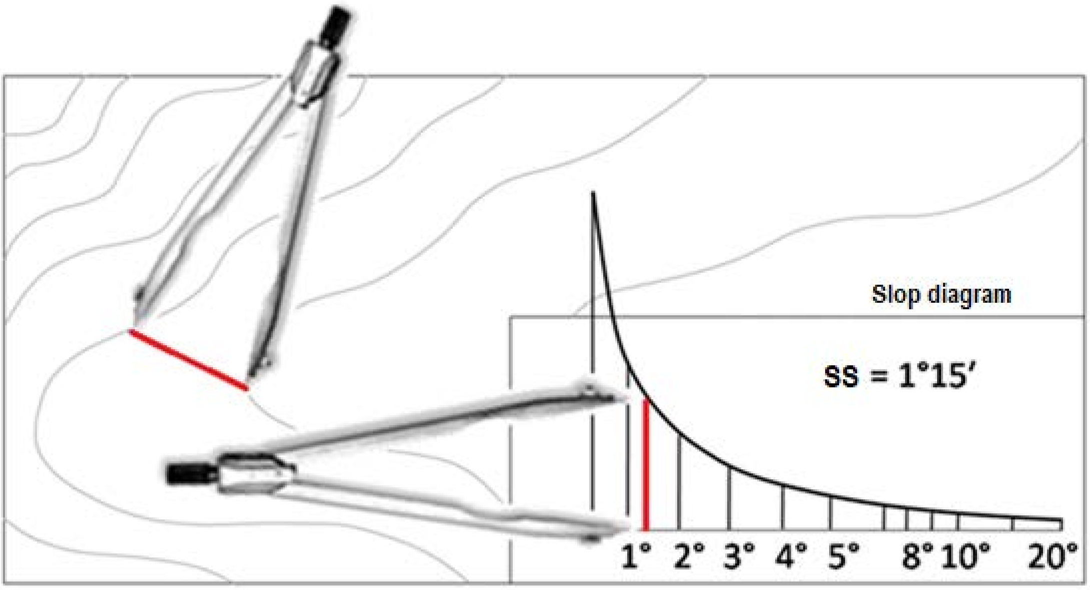


Fig. 16: Determining the slope angle from the slope diagram

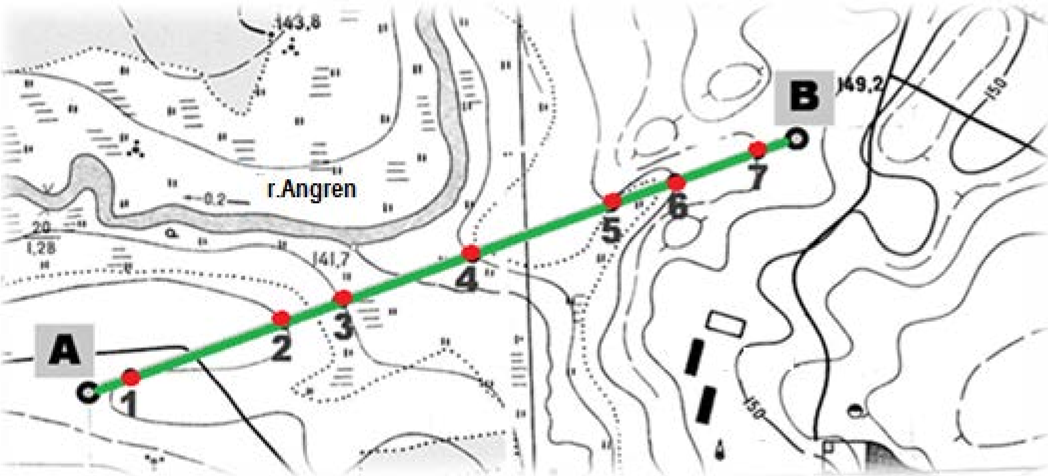
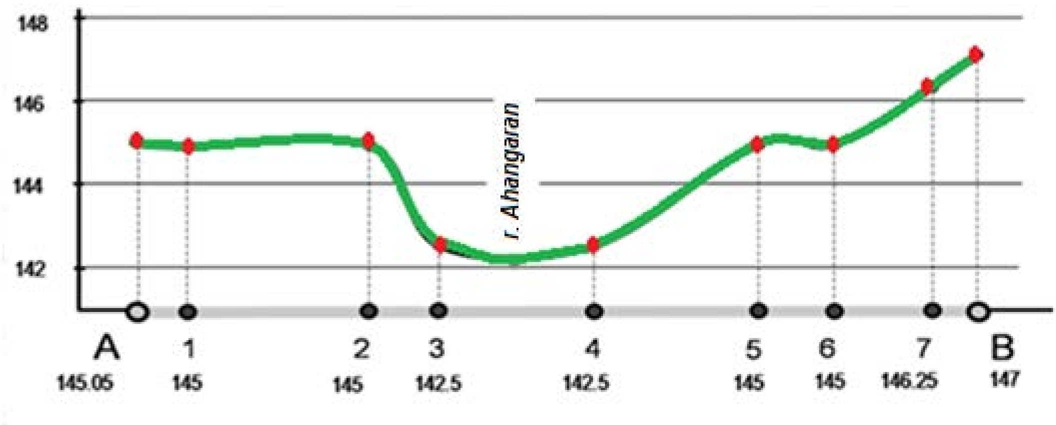


Fig. 17. profile line AB on the map

4.4 Construction of a profile along the AB line.
In order to build a terrain profile on the map, one should mark on the AB line points 1, 2, ..., n of its intersection with contour lines and other characteristic terrain lines (tops, watersheds, etc.). The heights of the intersection points are equal to the altitudes of contour lines or they are defined by interpolation (Fig. 17).
A horizontal line is drawn on a sheet of paper, the altitude of which is less than any of the intersection points, on which the intersection points are plotted to the scale of the map. Perpendicular segments are reconstructed from each obtained point at a vertical scale of 1:200. The ends of the built segments are connected by a smooth curve, which will represent the terrain profile.
The profile of the terrain along the line AB, shown in Fig. 17, is shown in Fig. 18



Scales: vertical 1:200, horizontal 1:10000

Fig. 18. Construction of a terrain profile along the AB line

4.5 One of the basic requirements in the design of roads and other communications is to keep the slope within specified limits while keeping the length of structures to a minimum.

LABORATORY ASSIGNMENT#2
STUDYING THEODOLITE 2T30M.MEASURING HORIZONTAL AND VERTICAL ANGLES

Measurements on the earth's surface are made to make connections between individual points and to determine their coordinates.
These measurements provide data to solve a variety of engineering problems, and measurement work is done by several established techniques and methods. To survey a polygon or broken line, you need to measure straight lines and horizontal angles of rotation of lines. To depict the terrain, the elevations of points are needed, for which the vertical angles are measured.
To measure horizontal, vertical angles and distances an angular measuring device - theodolite is used.
Purpose of work: to study the device of technical accuracy theodolite 2T30M (T-30) and tachymeter TS06 LIECA, to master the rules of use.
Perform calibration and alignment of theodolite.
Measure and calculate the horizontal and vertical angles.
Procedure of carrying out laboratory work.
Task 1.
Draw a scheme of geometrical axes of theodolite, showing the
all axes. 1.2.
1.2 List the names of the geometric axes of rotation
theodolite:
1.
2.
3.

4.

1.3 Write which geometric conditions the theodolite must meet.

The geometric conditions of the theodolite:

1.

2.

3.

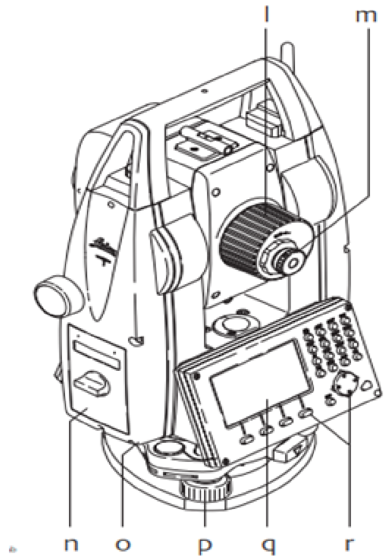
4.

Task 2.

2.1 Make a copy of figure 21,a,b,c,d and mark with numbers on it

The main parts and screws of theodolite 2T30M and total station TS06 LIECA.

2.2 Write down their names in order.



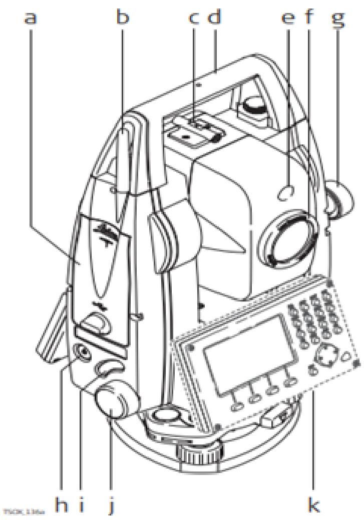


Fig. 21. a) Theodolite 2T30M

b, c) TS06 LIECA Total Station

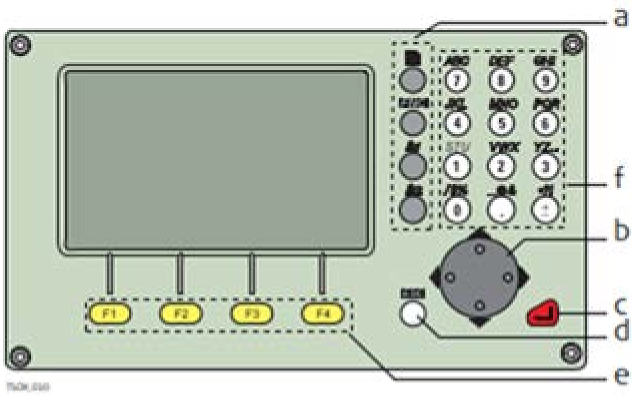


Fig. 21. d) TS06 LIECA Total Station

Task 3.

3.1 Learn the rules of taking readings on the scales of horizontal and vertical circles of theodolite 2T30M.

Figure the field of view of theodolite 2T30M indicating microscope scales (Fig.22) and write down the values of the resulted readings on vertical and horizontal circles

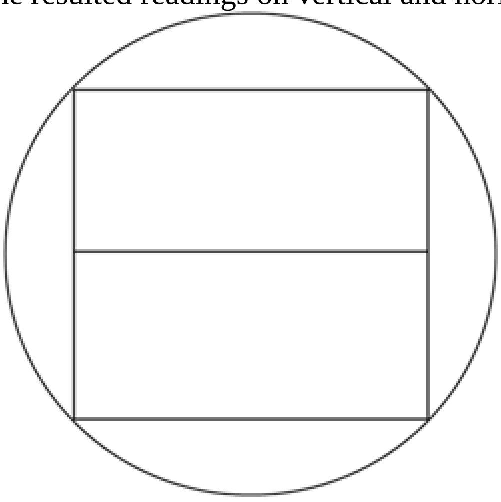


Fig. 22. Schematic diagram of the field of view of the theodolite scale microscope

Task 4.

4.1 Measure the horizontal angle.

Individually each student should measure and calculate the horizontal angle within the allowable error and record the results in the journal of field measurements of horizontal angles .

Measurement of the angle perform two half-measures, respectively, at the positions of "circle left" (CL) and "circle right" (CR).

No. reception	standing point	sighting point	CL KP	Horizontal circle countdown	Angle value from half-reception	Average value of the angle from the half-receive	Meaning horizontal angle
1	2	3	4	5	6	7	8
1	O	B A B A	CL CR				
2	O	B A B A	CL CR				

Task 5.

5.1 Measure the vertical angle.

Individually each student should measure and

calculation of the vertical angle within the allowable error and record the results in the vertical angle measurement log

No. standing point	sighting point	CL KP	Counting in a vertical circle	MO value	Meaning vertical angle
1	2	3	4	5	6
O	B	CL CR			

. 5.2 Determine the "place of zero" (**MO**) and the slope angle (v).

The MO zero place and the slope angle are determined by the formulas:

$$\mathbf{MO} = (CR + CL) / 2; \qquad (19)$$

$$v = (CL - CR) / 2; \qquad (20)$$

$$v = CL - MO = MO - CR. \qquad (21)$$

The vertical angle or tilt angle is the angle in the vertical plane between the tilt line of sight and its projection onto the horizontal plane.

The quality control of vertical angle measurements is the constancy of the MO when observing at different sighting points.

For 2T30M type theodolites permissible value of the zero point fluctuations is ~ ± 2′.

Methodical instructions

Theodolite 2T30M is intended for measuring horizontal and vertical angles in mine workings and on the surface as well as for measuring distances by the telescope telescope string-meter.

Theodolite technical optical surveying 2T30M is a repeating type instrument with a scale reference microscope.

The telescope with both ends is transferred through a zenith. It is focused on the object by turning the focusing ring.
The diopter eyepiece ring of the telescope establishes a clear image of the reticle.
The telescope sight serves for preliminary aiming of the telescope at the target. The exact aiming in the vertical plane is done by aiming screw of the vertical circle when the fixing screw is clamped.
The theodolite is rotated and the telescope is pointed accurately in the horizontal plane by the aiming screw of the horizontal circle with the fixing screw clamped.
Angle readings are taken from the scales of the horizontal "D" and vertical "B" circles. The image of counting devices with the help of optical system is summarized in the field of view of the counting microscope (Fig. 23).

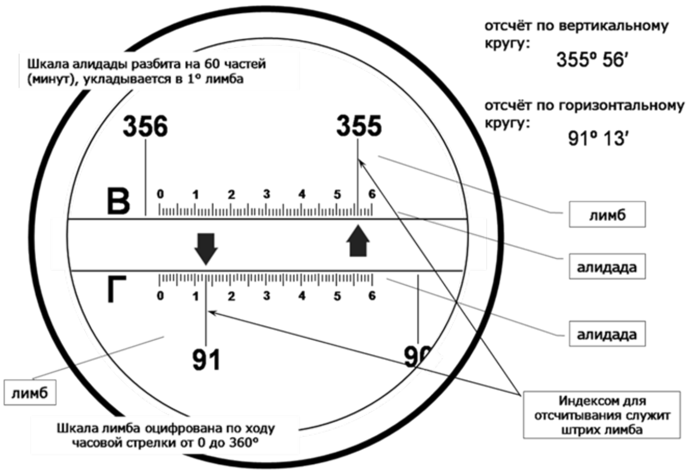
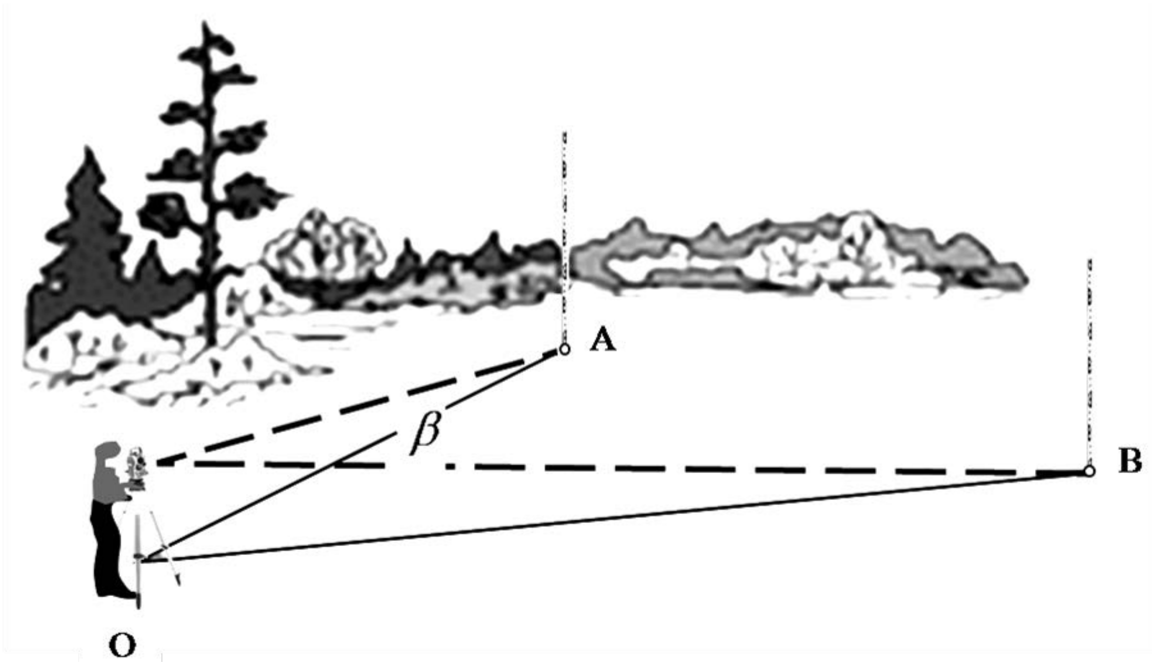


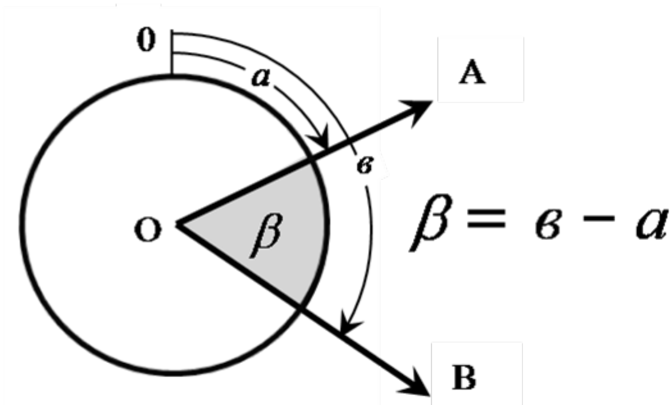
Fig. 23. field of view of the scale microscope of theodolite 2T30M

The axes and planes of the horizontal and vertical circles are adjusted to their respective positions using the cylindrical level located on the theodolite body.
The 2T30M theodolite has glass limbs with digitised graduations every 1°. The scale of the alidade is divided into 60′ and digitised in 10′. There are two scales in the microscope's field of view. The upper part of the scale, marked "B", has a vertical circle limb, the lower part of the scale, marked "D", has a horizontal limb. The readout can be taken with an eye to half or quarter of a graduation (30″ or 15″) (fig. 23).
Measuring the horizontal angle using the technique
To measure the horizontal angle AOB, the theodolite is set
at the top of the corner at point O, centre it with a plumb bob at least 2mm and with the aid of levelling screws and a cylindrical spirit level bring it to the working plane (fig. 24).
Measurement will begin at the CL. Using the telescope, aim the telescope at point A and screw in the fixing screw. The crosshairs of the reticle are precisely aligned to the target (point A) by means of the pointing screw. Look through the microscope of the reticle and, by changing the position of the illumination mirror, achieve the best possible visibility of the horizontal and vertical circle scales. Read off the scale of the horizontal circle and record in column 5 of the field log of horizontal angles (Table 1).
The telescope is aligned with point B after removing the alidade with the screw fixing the horizontal circle and, after fixing the screw, the crosshair of the reticle is precisely aligned with the aiming screw. Take a reading and enter it in column 5 (table 1).
The horizontal angle AOB ( ) is the difference between the coordinates of point B and point A:
AOB = c - a, (22)

If the horizontal circle datum B is less than the datum A, add 360º to it (Fig. 25).
To measure the right circle angle (KP), the pipe is moved across the zenith and the above actions are repeated in the same sequence. All points A and B of the right circle are set by means of the fixing and pointing screws. The control readout is recorded in column 5 (Table 1) and the values of the half-acceptance angle are calculated in column 6, the average value of the acceptance angle in column 7 and the final value of the angle in column 8.



- Fig. 24: Measuring the horizontal angl



- Fig. 25: Horizontal angle measurement diagram

Table 3

№ стан-ции	Поло-жение круга	Точка на-блю-дения	Отсчёты по горизонтальному кругу	β	$\beta_{ср}$
I	КЛ	B	231⁰46ˡ	64°45′	64°44′30″
		A	148⁰56ˡ		
	КП	B	233⁰20ˡ	64°44′	
		A	78⁰36ˡ		

After comparing the measured angles at CL and CR, determine their discrepancy, which must not exceed double the accuracy of the angle measurement with this theodolite (Table 3).
For 2T30M - 1′, if the discrepancy exceeds 1′, the measurements are made again.
Measurement of the vertical angle
The vertical angle is measured with the vertical wheel at CL and CR.
By acting on the pipe clamping screw and the guide screw,
The centre horizontal strand is first aligned with one circle (CL) to point A and measured against the limb of the vertical wheel, then the centre line is moved across the zenith by detaching the alidade of the horizontal wheel. The centre line is aligned to point A at the control and the readout is taken. The reading is recorded in column 4 of Table 2 for vertical angle measurements.
When pointing the pipe at the target, observe the position of the bubble of the cylindrical level at the alidade of the horizontal circle, which must be at the zero point both at CL and CR.
The values of the readings of the two circles CL and CR are used to calculate the MO - zero point of the vertical circle and the angle of inclination of the line according to the formulas (19), (20) and (21).
Fluctuations in the MO when measuring vertical angles should not exceed the value of the error of measurement of the vertical angle by one method, equal to ≈ 2′.

LABORATORY ASSIGNMENT #3

STUDYING NIVELIR AND PERFORMING ITS VERIFICATIONS _ _ _ _

A leveller is an instrument designed to determine the elevation between two points on the earth's surface by means of geometric levelling using a horizontal beam and staff.

Purpose of work: to study the construction of the NA332 LIECA leveller, to learn how to read off a levelling rod and to measure distances with a corded distance measurer.

Carry out calibration and alignment of the leveller.

Order of carrying out laboratory work.

Task 1.

1.1. draw a diagram of the geometric centreline of the levelling rod, showing the mandatory mutual alignment of the centreline and the centreline.

allocation of the axes.

1. ____
2. ____
3. ____
4. ____

1.2**.** List the names of the geometric axes of rotation of the **level**
:
1.
2.
3.
4.

1.3 Write down which geometric conditions must be satisfied by the leveller.
The geometric conditions of the leveller:
1.
2.
3.

Task 2.
2.1 Copy figure 26 and number the main parts and screws of the NA332 LIECA leveller.
the main parts and screws of the NA332 LIECA.
2.2 Write down the names of the NA332 LIECA parts and screws in order.

Rice. 26. Level **NA332 LIECA** and staff

Task 3.
3.1 Learn how to read off a levelling staff and measure
Measure distances with a string reticle (Fig. 27). Give your own representation of the field of the leveling telescope, showing part of the levelling staff, the reticle and the bubble of the cylindrical level.
Calculate the distance from the leveller to the staff using the formula:
I = (CH - NE) · k = , (23)
where k is the rangefinder coefficient, k = 100;
CH - readout from the bottom line of the cross of the reticle;
CB - readout by the upper thread of the thread grid cross;
a - readout by the middle thread

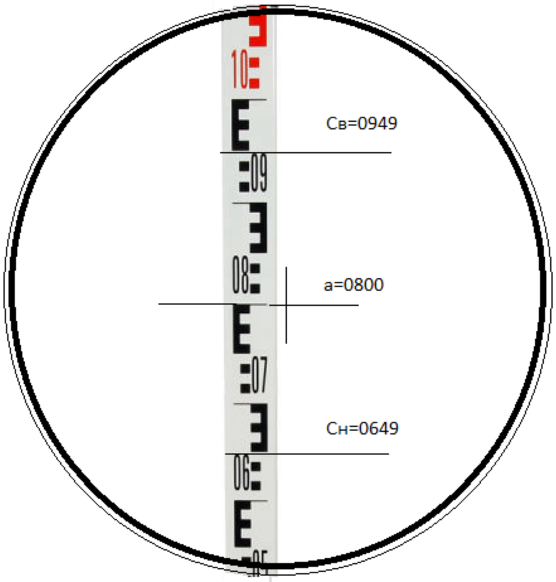


Fig. 27. View of the NA332 LIECA field of view

Reference line readings: NE = 0949 mm; CH = 0649 mm; a = 1250 mm,
I = (CH - NE) · k = 30 m.

Task 4.

4.1 Carry out checks on the circular level, the grid position and the the main condition of the leveller. Answer the question as to whether the leveller needs adjustment.

Calibrate the circular level .... ( );
verification of the reticle position .... ( );
verification of the main levelling condition .... ( ).
In brackets indicate which geometric condition is being checked._

4.2 Record the results of the main levelling condition in Table 4. Calculate the exceedance (h1-2) and the error x in the double levelling using formulae (24) to (30): taking the error into account,

$$h_1 = h_2\ (h_{1\text{-}2}); \qquad (24)$$
$$h_1 = i_1 - a_1; \qquad (25)$$
$$h_1 = i_1 + x - a_1; \qquad (26)$$
$$h_2 = a_2 - i_2; \qquad (27)$$
$$h_2 = a_2 - i_2 - x; \qquad (28)$$
$$i_1 + x - a_1 = a_2 - i_2 - x; \qquad (29)$$
$$i_1 + x - a_1 = a_2 - i_2 - x; \qquad (30)$$

4.3. Draw a double levelling diagram explaining the values a_1, a_2, i_1, i_2, x.

Table 4

station number	height , mm	Readings for rail a , mm	Excess _ _ h , mm	Error, x , mm	Correct countdown _ a'_2 , mm
1	$i_1 =$	$a_1 =$			
2	$i_2 =$	$a_2 =$			

Right counting off at i_2 calculates c i according to the form :

$$a'_2 = a_2 - x . \qquad (31)$$

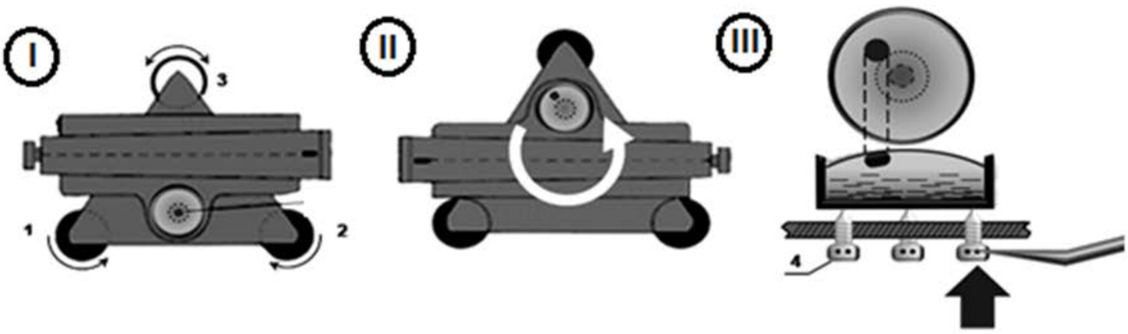
Correction _ _ cylindrical u ro v nya produce in clothes, es l and $X \geq$ four mm .

Methodical instructions _

The NA332 LIECA is designed for geometric levelling. In working position, the sighting axis is horizontal. It is a cylindrical level with a telescope in its horizontal position.
The main requirement for the NA332 LIECA is that the telescope axis is parallel to the telescope axis.
Make sure that the instrument is suitable for levelling before you start work. The functionality of the individual parts of the instrument and the correct alignment of its axes must be checked
is checked by means of calibrations. If there are any deviations, you should correct them, i.e. adjust the leveller.
Bringing the leveller to working position (setting the axis of rotation of the leveller in a plumb position) is carried out by lifting screws on the circular level. 1.

The axis of the circular level must be parallel to the axis of rotation of the leveller (U1 U1 || TT).
Turn the lifting screws to bring the level bubble to the centre. The upper part of the leveller is rotated 180°. If the bubble does not move, the condition is fulfilled. Otherwise the level screws move it to the zero point by half the bend and then the lifting screws move it to the centre. Repeat the calibration and alignment (fig. 28).

2. The vertical stroke of the reticle should be parallel to the axis of rotation of the leveller and the horizontal perpendicular to the axis (in || TT, yd TT).
Bring the axis of rotation of the leveller to a plumb position. Aim the vertical line of the reticle to the plumb line located at a distance of 20 - 25 m from the leveller. If the vertical line of the reticle does not coincide with the plumb line, the condition is not fulfilled.

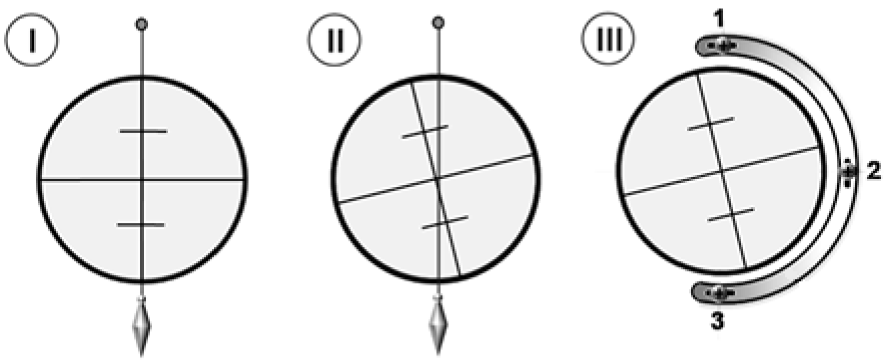


Rice. 28. Checking the round level of the level

In this case, remove the eyepiece part of the tube and open the frame
with the reticle, the frame is screwed down with three screws. Loosen the upper and lower screws a full turn and the middle one a quarter turn. Then turn the plate in the desired direction. Put on the eyepiece part of the tube and check the position of the vertical string. When it is in place, fasten the middle and then the upper and lower screws of the reticle frame and the eyepiece piece of the tube (Figure 30).

The verification can also be carried out in another way where the condition "the horizontal thread of the reticle must be perpendicular to the axis of rotation of the instrument" is checked.

The telescope is pointed at a clearly visible point and by rotating the
the point is moved to another position by turning the telescope guide screw. If the point is moved away from the horizontal line, the calibration is deemed to be
unaccomplished and if it has happened in the field and the work has to be completed, it is recommended to read off from the reticle by referring to the centre of the reticle
of the reticle.



Rice. 29. Checking the mesh of threads _ _ _

3 The axis of the cylindrical level must be parallel to the
to the sighting axis (U2U2 || VV).

This is otherwise known as: verification of the main levelling condition**.**

This condition is verified by double levelling forward the two points 1 and 2, fixed at a distance of 30 - 40 m from each other on a line 1-2 (Fig. 31).

The leveller is set over point 1 and in working position, its height i1 is measured with a levelling rod over point 1. Then sighting onto the staff set over point 2 and, having zero-pointed the level bubble with elevation screw (by matching the ends of the bubble in the sight of the tube), a1 is read off from the staff (equal to the true reading a′1 plus error x due to the lack of parallelism between the levelling sight and the level axis).

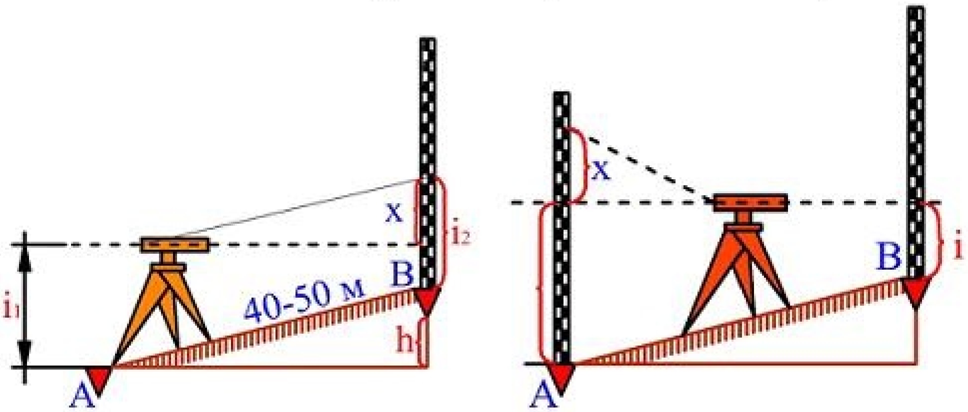
The excess (h1-2) between points 1 and 2 is calculated as in the case of "levelling forward" by formula (25), taking into account the error (26) where x is the error of readout because between the longitudinal axis and the level axis exists the angle " " which at the time of verification is unknown and it needs to be defined.

Level and a rail are interchanged places, bring the device in an operating position, measure a new height of device i2 and take readout on a rail and a2 (which in the sum is equal to true readout a2′ and error x - owing to nonparallelism of a sighting axis and a level axis):

$$a = a' + x; \ (31) \quad a' = a - x. \ (32)$$

The excess between points 1 and 2 is found by formulae (27), (28) and further, using formulae (29), the value of x is found by formula (30).

The error caused by non-compliance with the main levelling condition must not exceed 4 mm in absolute value.



x is the error due to the inclination of the sighting axis to the horizon

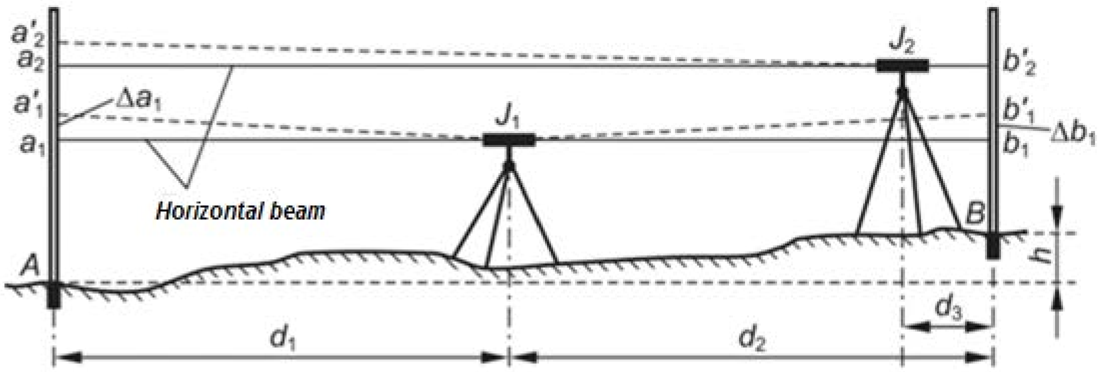


Fig.30. Checking the main condition of the level

Now, knowing ***x*** , we can define divide _ true ots h et

$$a_1' = a_1 - x; \qquad (33)$$

$$a_2' = a_2 - x, \qquad (34)$$

when _ _ _ _ horizontal . _ _ _

In case of non - compliance with the main condition of the N-3 level , adjustment is made . Corrections ***are*** made , if **_ ≥ 4 mm** .

If necessary , y can also be calculated for the angular value of the error.

The value of g α is calculated by the formula :

$$\alpha'' = (x / S) \cdot \rho'', \qquad (35)$$

where ***S*** – horizontal distance between **_ _ 1** and **2** ;
ρ'' is the number of seconds in radian ; _
α'' - at the goal between the sighting axis and the axis of the cylindrical at the level , expressed in seconds .

Angle α'' must not exceed 20 ′ ′ .

If the calculated value of x does not exceed 4 mm, the

practically fulfilled.

If this is not the case, the instrument is adjusted without removing it from the second station

without removing the second station to calibrate the instrument.

To do this, use the elevation screw to set the

the horizontal line of the reticle to a reference point on the staff which equals

$a' = a - x$.

This will separate the images of the ends of the level bubble. Slightly loosen the side levelling screws on the cylinder level by means of the vertical levelling screws (rotating them in opposite directions) to achieve the alignment of the bubble ends. Then fasten the side alignment screws.

Repeat the checking procedure.

TEST QUESTIONS

Subject and tasks of geodesy.
The concept of the shape and size of the Earth.
Level surface properties.
Krasovsky's reference ellipsoid.
The principle of depicting the earth's surface on a plane.
Geographical coordinates.
Determination of rectangular coordinates on a topographic map.
Determination of geographical coordinates on a topographic map.
Heights of points on the earth's surface. Absolute, conditional point height.
Determination of elevation on a topographic map.
The concept of the Baltic height reference system.
The concept of the Gauss-Kruger rectangular coordinate system.
orienting angles. Relationship between orienting angles.
Determination of the directional angle on a topographic map.
Methods for determining the position of points on the ground.
The concept of a plan, map, profile, section.
Conventional signs and their classification.
Representation of relief on plans and maps.
Brief information about the construction of geodetic networks.
Types of topographic surveys.
Solution of direct geodetic problem.
Solution of the inverse geodesic problem.

Checking and adjusting theodolite. Geometric conditions of theodolite.
Level checks and adjustments. Geometric conditions of the level.
The main condition of the level.
Measurement of horizontal and vertical angles.
Collimation error and "zero point".
The principle and methods of geometric leveling.
Production of tacheometric survey.
Surface leveling.
Difficult leveling.
trigonometric leveling.
Determination of the value characterizing the steepness of the slope and the slopes of the lines.
Scheme for constructing a terrain profile in a given direction.
Show the field of view of the level
Determining the place of zero (MO).
Office processing of field measurements.
True and magnetic azimuths. Orientation chart on the map.
Tacheometric survey.
Total station LEICA TS 06. (structure, functions, accuracy, etc.)
Level NA 332 (structure, functions, accuracy, etc.)
Reiki for leveling.

12. GLOSSARY OF GEODETIC TERMS

Absolute height of a point on the earth's surface (altitude) - The vertical distance (usually in meters) from this point to the average level of the ocean surface. In the Russian Federation, it is calculated from zero footstock in Kronstadt.
Angle of inclination (steepness of the slope) - The vertical angle measured from the horizontal plane up from 0° to +90°, and down from 0° to -90°.
Angle of rotation of the track - An angle with a vertex formed by the continuation of the directions of the previous side and the direction of the next side.
Adjustment - A set of mathematical operations performed to obtain the most probable value of the geodetic coordinates of points on the earth's surface and to assess the accuracy of measurement results.
Altitude network - A network of points on the earth's surface, the heights of which above sea level are determined from leveling. Leveling network points are fixed on the ground with leveling marks and benchmarks, which are laid in the walls of durable structures or directly into the ground to a certain depth. The leveling network serves as a high-altitude basis for topographic surveys, and when leveling heights are re-determined, its points are also used to study the vertical movements of the earth's crust.
Autocollimator - An optical device for accurate angular measurements, control of straightness and parallelism of planes, using the principle of autocollimation. It is used in electronic theodolites, tacheometers.
Automated cartography - A section of cartography that covers the theory, methodology and practice of creating, updating and using maps, atlases and other space-time cartographic works in graphic, digital and electronic forms using automatic cartographic systems and other technical and hardware and software tools.
Azimuth - The angle between the north direction and the direction to any given object. Azimuth is usually measured in the direction of the apparent movement of the celestial sphere (clockwise on maps)
Azimuthal projections - Cartographic projections, the parallels of the normal grid of which are concentric circles, and the meridians are their radii, diverging from a common center of parallels at angles equal to the difference in longitudes. Normal azimuth projections are used

for maps of polar countries, transverse and oblique azimuthal projections are used for maps of the earth's hemispheres, continents, the starry sky, the Moon and other planets.
Alidada - Device for measuring angles (rotating around an axis passing through the center of the limb) in astronomical, geodetic and physical goniometric instruments.
Almanac of navigation satellites - A set of reference information about the position (on the time scale and orbital elements) and the operational status of all NS of a given GNSS, included in the information transmitted from the satellite.
Altimeter - A device that measures atmospheric pressure to determine absolute and relative heights.
Angalif color method - (from the Greek. anaglyphos - relief), obtaining a stereoscopic (volumetric) image using 2 images painted in complementary colors that make up a stereo pair, viewed through differently colored filters (multi-colored glasses). It is mainly used to create three-dimensional illustrations in textbooks, for three-dimensional representation of relief on geographical and geological maps, etc.
Astrolabe - A goniometer used to measure horizontal angles and determine latitudes and longitudes in astronomy.
Atlas - A systematic collection of maps with explanatory text, published in the form of a volume or a set of separate sheets (for example, a geographical atlas, an astronomical atlas).
Aerial filming - Surveying the terrain from aircraft using surveying systems (information receivers) operating in different parts of the spectrum of electromagnetic waves. There are photographic, television, thermal, radar and multi-zone aerial photography. Read more http://tochno-rostov.ru/topograficheskaya-semka/ortofotoplan/
Aerial photogrammetry - A section of photogrammetry that studies how to measure various objects from aerial photographs.
Aerial photography - Photographing (in all ranges of the optical spectrum) terrain from an aircraft. Distinguish between planned and prospective aerial photography. Aerial photography materials are used in geodetic, geological surveys, engineering surveys, etc. In our company, we perform this work using unmanned aerial vehicles http://tochno-rostov.ru/topograficheskaya-semka/ortofotoplan/
Aerial topography - A section of topography that studies methods for creating topographic maps based on aerial photography.
Approximation of meridians - The angle that forms a tangent to the image of any meridian with the first coordinate axis (abscissa) of this projection, which is usually the image of the middle (axial) meridian of the displayed territory.
Arbitrary projections - Map projections that distort angles and areas. There are equidistant, preserving the scale of lengths in one of the directions (for example, along the meridians or parallels), and orthodromic, in which the great circles of the ball (orthodromes) are depicted as straight lines. Used for world maps.

B

Basis - A line directly measured on the ground with high accuracy and intended both for comparing and studying measuring instruments, and for determining the lengths of the sides of a geodetic network.
Beam - An elongated depression with soddy slopes of various steepness, up to several tens of meters deep.
Baltic system of heights - The accepted system of absolute heights, which are counted from the zero of the footstaff in Kronstadt.
Barometric leveling - A method of approximately determining the difference in altitude between two points by the values of atmospheric pressure at these points.
Bathymetric maps - (from the Greek bathys deep and metreo I measure), display the underwater relief using isobaths together with depth marks.

Bathymetry - Measurement of the depths of water bodies (seas, oceans, lakes, rivers, etc.) using special instruments of various systems (lots, echo sounders, etc.).
Bergstrokes (slope markers) - Short strokes on the contour lines of a topographic map, indicating the direction down the slope.
Benchmark - A sign of a point with a known absolute height - a metal disk with a ledge (or with a hole - a mark), fixed in the walls of long-term structures, or a concrete monolith laid in the ground.
Bisector - Two strokes of a grid of filaments of a spotting scope in geodetic instruments, used together to aim at a sighting target.
Block diagram - Perspective image of any part of the earth's surface together with a geological section of the earth's crust. It is used mainly in the study of the relationship between relief and geological structure.
Bussol - An instrument for measuring the magnetic azimuth of directions on the ground. They are used in geodetic works, in mine surveying.
Building grid - A system of squares and rectangles with sides of 100-200 m parallel to the main axes of structures. It is designed on the master plan, placing the sides of the figures as close as possible to the designed structures. For the origin of coordinates of the conditional system, for the convenience of calculations, a grid point located in the southwestern corner of the construction site is chosen.
Bearing - The angle between the direction to the observed object and one of the main planes taken as the origin of the angular coordinates. In maritime and air navigation, usually the same as azimuth.
Base - Geodetic construction at the construction site, providing the definition of elevation marks of the design elements of the complex. Elevations of points of the high-altitude center base are determined by leveling of the IV class. The points of the state leveling network (GNS) are supplemented with construction benchmarks from the calculations of at least 2 for each construction site, and for multi-section buildings, one construction benchmark for each station.

C

Cadastre - A systematized set of basic information about certain natural objects.
Cameral tracing - Designing a route according to topographic maps, plans, aerial survey materials and digital terrain models.
Cartogram - A map showing by shading (of varying density) or coloring (of varying degrees of saturation) the average intensity of any indicator within each unit of the territorial division plotted on the map.
Cartography - A section of cartography that studies the processes, methods and technologies for creating cartographic works.
Cartographic grid - The image on the map of geographic meridians and parallels in one or another cartographic projection. It serves to build a cartographic image and allows you to determine the coordinates of points on the map.
Cartometry - A section of cartography that studies how to measure various geographical objects on maps to obtain their quantitative characteristics.
Compass - A device that indicates the direction of a geographic or magnetic meridian; serves for orientation relative to the sides of the horizon. There are magnetic, mechanical (gyrocompass), radio compass (direction to the radio beacon), etc.
Conic projections - Cartographic projections, the parallels of which are the arcs of concentric circles, and the meridians are their radii, the angles between which are proportional to the differences in longitude; distortions of conic projections do not depend on longitude. Used for maps of territories stretched along parallels (for example, the Russian Federation).
Coordinate grid (topographic) - A set of two families of mutually perpendicular lines drawn parallel to the axes of rectangular coordinates and forming a rectangular grid.

Coordinator - A device for measuring the coordinates of points (indicative targets, etc.) on topographic maps with a rectangular coordinate grid, as well as for plotting points on maps using known coordinates.
Coordinates - Numbers, the assignment of which determines the position of a point on a plane, surface or in space.
Caliper - Drawing compasses, in which the angle between the legs is set and fixed with a micrometric screw.
Curvimeter - A device for measuring the length of curved lines on topographic maps and plans.
Combined survey - A method of creating topographic maps of flat-plain populated areas, in which the contour part of the map is obtained from aerial photographs or photographic plans, and the relief is reproduced on aerial photographic material in the field using scale survey techniques.
Celestial sphere - An imaginary surface of arbitrary radius centered at an arbitrary point on which heavenly bodies are projected.
Centering - An operation to align the vertical axis of the measuring instrument with a plumb line passing through the reference point of geodetic measurements.
Cylindrical projections - Cartographic projections, the meridians of which are equidistant parallel lines, and the parallels are lines perpendicular to them. They are used to depict areas elongated along the equator or some parallel. In navigation, the G. Mercator projection is used, and when creating topographic maps, a conformal transverse cylindrical projection is used.
Compasses - A drawing tool for measuring segments on a map (plan) or aerial photograph with their simultaneous increase and decrease in the required number of times.
Coordinate system - A set of mathematical rules that describe how coordinates should be related to points in space.
Conformal projections (conformal projections) - Cartographic projections that convey angles on maps without distortion and maintain a constant scale in all directions at each point, although the scale is different in different places on the map. Used to build large - and medium-scale maps.
Condensation geodetic networks (networks of local importance) - They are created during the development of a geodetic network of a higher order (class). They serve to increase the density of the state network, based on the needs of the assigned engineering and geodetic tasks.
Catchment - A line on the earth's surface from which surface and groundwater drain into a specific body of water.
Contours (isohypses) - Closed curved lines on the map connecting points of the earth's surface with the same absolute height and together conveying landforms.
Closed polygon - Geodetic construction on the ground in the form of broken lines forming a closed geometric figure.

D

Detailed staking of curves - Stakeout of the picket *route* on a circular curve and assignment of curve radii.
Deformation of structures - A change in the relative position of the entire structure or its individual parts, associated with spatial movement or a change in its shape.
Deciphering - The study of aerial photographs and space images of territories, based on the relationship between the properties of decipherable objects and the nature of their reproduction in the images.
Depth zero - A conditional surface from which depth marks are given on marine navigational charts. The established zero depth in the Russian Federation for seas without tides (Baltic, etc.) and lakes is the average long-term level; in seas with tides (Okhotsk, etc.) - the lowest sea level, derived from the data of level observations.
Directional angle - Flat angle between the northern direction of the line parallel to the axial meridian of the zone and the direction of the object; counted clockwise from 0 to 360°.

Discrepancy - The difference between the resulting value and the one that should be.
Direction finding - Determining the direction to any object - its angular coordinates. It is carried out by optical, radio engineering, acoustic and other methods.
Deviation of plumb lines - An angle formed when a plumb line drawn at a point on the earth's surface perpendicular to the geoid does not coincide with a normal drawn at the same point perpendicular to the ellipsoid.
Doppler geodetic network - Represented by 131 points, the mutual position and coordinates of which are determined by Doppler observations of artificial satellites of the Earth of the "Transit" system.
Direct geodetic problem - Calculation of geodetic coordinates - the latitude and longitude of a certain point lying on the earth's ellipsoid, according to the coordinates of another point and the known length and directional angle of a given direction connecting these points.
Design slope - The tangent of the slope of the design line or plane.
Digital terrain model - A digital cartographic model that contains data about terrain features and its characteristics.

E

Engineering geodesy - A branch of geodesy that studies the methods of measurement and tools used in engineering surveys and the construction of engineering structures. Components of engineering geodesy: topographic and geodetic surveys, engineering and geodetic design, marking work, alignment of structures, monitoring of deformations of structures.
Engineering and geodetic design - A set of works carried out to obtain the data necessary to place the structure in plan and in height.
East (point of the east) - The point of intersection of the mathematical horizon with the celestial equator, lying to the right (in the middle between the points of north and south) from an observer facing north; denoted V., O (German Ost) or E (English East).
Earth 's rotation is the rotation of the Earth around its axis from west to east, or counterclockwise as viewed from the North Pole of the World. The rotation of the Earth causes a change of day and night, determines the length of the day. Occurs unevenly: under the influence of mainly lunar and solar tides (tidal friction), the duration of the day continuously increases by 1-2 ms per century, and due to seasonal changes (precipitation, etc.), tectonic processes and others during the year fluctuates within 1-2 ms. The position of the Earth's axis of rotation, and hence the Earth's geographic poles, changes due to precession and nutation.
Edge - A line located on the verge of transition of a slope of lesser steepness into a slope of greater steepness.
Eye survey - A simplified topographical survey carried out using a light tablet, a compass and a target line to obtain an approximate plan of a route or area of area.
Elevation - Same as relative height.
Earth's magnetic poles - Points on the earth's surface where the magnetic needle is located vertically, i.e. where the magnetic compass is not applicable for orientation to the cardinal points.
Eker - A portable geodetic instrument for determining the planned position of points by constructing angles on the ground that are multiples of 90° or 45° (prismatic and box-shaped akers) or equal to 90° (two-mirror akers). It is used when shooting small areas of the area.
Eklimeter - A portable geodetic device for measuring the angles of inclination on the ground.
Ecliptic - A large circle of the celestial sphere, inclined to the celestial equator at an angle of approximately 23 ° 27 ', along which the center of the Sun moves in its apparent annual movement, reflecting the movement of the Earth in its orbit.
Electronic map - A digital cartographic model generated on a machine medium using software and hardware (*GIS*) in
the accepted projection, the system of coordinates and heights, conventional signs intended for display, analysis and modeling, as well as solving information and calculation problems based on terrain and situation data.

Epoch (navigation satellite) - The moment in time at which the satellite is at some point in the orbit.
Equator - A line of section of the earth's surface by a plane passing through the center of the Earth, perpendicular to the axis of its rotation. Divides the globe into northern and southern hemispheres. Serves as the beginning of the calculation of geographic latitude. The length is about 40,076 km.
Error theory - A section of mathematical statistics devoted to the construction of refined conclusions about the numerical values of approximately measured quantities, as well as about errors (errors) of measurements.
Equivalent projections (equivalent) - Cartographic projections that preserve a single scale of areas throughout the map, due to which the areas of the figures on the map are proportional to the areas of the corresponding figures in nature; used in small scale constructions.

F

First vertical - The plane perpendicular to the meridian.
Focal length - The distance from the main focus of the lens to its optical center.
Field tracing - Transfer of the designed route to the terrain with clarification of its changes and fixing in nature.
Fundamental catalogs - Star catalogs that fix the fundamental system of celestial equatorial coordinates in the sky with maximum accuracy - the basis for studying the movements of celestial bodies and determining astronomical coordinates, time and azimuth for points on the Earth's surface.
Field of view (satellite) - A section of the earth's surface from which observation of the satellite is possible (reception of signals from the satellite at a given time)

G

Gauss-Kruger projection - A conformal cartographic projection in which topographic maps of Russia and some other countries are compiled.
Gaussian convergence of meridians - The angle between the geodesic meridian of a given point and a line parallel to the axial meridian of the coordinate zone.
Geodetic quantity - A physical quantity to be measured in the process of geodetic work. For example, horizontal angle, length, coordinate increment, etc.
Generalization - Generalization of geographic images of small *scales* relative to larger ones, carried out in connection with the purpose, subject matter, knowledge of the object or technical conditions for obtaining the image itself.
Geographic basis of maps - General geographical elements of a thematic map that are not included in its special content and facilitate orientation and understanding of the patterns of placement of phenomena related to the subject of the map.
Geographic grid - A set of meridians and parallels on the theoretically calculated surface of the earth's ellipsoid, ball or globe.
Geographic Information Systems (GIS) - An information system that operates with spatial data.
Geographical maps - Maps of the earth's surface, showing the location, condition and relationships of various natural and social phenomena, their changes over time, development and movement. They are divided by territorial coverage (world, continents, states, etc.), by content (general geographical and thematic), by scale - large - (I: and larger), medium - (from I: and to I: I inclusive) and small-scale (smaller than I:I, as well as by purpose (reference, educational, tourist) and other features.
Geographical coordinates - Latitude and longitude, determine the position of a point on the earth's surface. Geographic latitude - the angle between the plumb line at a given point and the plane of the equator, measured from 0 to 90 ° on both sides of the equator. Geographic

longitude - the angle between the plane of the meridian passing through a given point, and the plane of the initial meridian. Longitudes from 0 to 180 ° east of the beginning of the meridian are called eastern, to the west - western.
Geodetic base - The geodetic base in the production of engineering and geodetic surveys at construction sites are: - GGS points (planned and high-rise); - points of the reference geodetic network, including geodetic networks for special purposes for construction; - points of the geodetic center base; - points (points) of the planned-altitude survey geodetic network and photogrammetric concentration.
Geodetic survey network - A network of condensation created for the production of topographic surveys. Subdivided into planned and high-rise.
Geodetic signs - Ground structures (in the form of pillars, pyramids, etc.) and underground devices (concrete monoliths), which designate and fix geodetic points on the ground.
Geodetic instruments (geodetic instruments) - Mechanical, opto-mechanical, electro-optical and radio-electronic devices used for the production of geodetic measurements.
Geodetic reference data - Geodetic coordinates of the reference geodetic network origin point, geodetic azimuth of the direction to one of the adjacent points, determined astronomically, and the height of the geoid at this point above the surface of the accepted earth ellipsoid. In the Russian Federation, the center of the round hall of the Pulkovo Astronomical Observatory is taken as the starting point, here the height of the geoid above the ellipsoid is considered to be zero.
Geodetic coordinates - The latitude and longitude of a point on the earth's surface, determined by geodetic measurements of the distance and direction from a point with known geographical coordinates, and the height of the point relative to the so-called. reference ellipsoid.
Geodetic quantity - A physical quantity to be measured in the process of geodetic work. For example, horizontal angle, length, coordinate increment, etc.
Geodetic point - A point on the earth's surface, the position of which in a known system of planned coordinates is determined by geodetic methods (triangulation, polygonometry, etc.) and fixed on the ground with a geodetic sign.
Geodetic satellite receiver - A receiver that provides reception of code-phase information transmitted from a satellite, designed to perform geodetic work.
Geodesy - The science of determining the shape, size and gravitational field of the Earth and measurements on the earth's surface to display it on plans and maps, as well as for various engineering and economic activities.
Geoid - The figure of the Earth, bounded by a level surface, continued under the continents.
Geo -image - Any spatio-temporal, large-scale, generalized model of terrestrial objects or processes, presented in a graphic figurative form.
Geoinformatics - A scientific and technical direction that combines the theory of digital modeling of a subject area using spatial data, technologies for the creation and use of geoinformation systems, the production of geoinformation products and the provision of geoinformation services.
Geoinformation mapping - Automated creation and use of maps based on GIS and databases of cartographic data and knowledge.
Geo-information space - An environment in which digital geo-information and geo-images of various types and purposes function.
Geoinformation resources - A set of banks (databases) of cartographic and thematic information.
Geoinformation technologies (GIS-technologies) - A set of techniques, methods and methods of using computer technology, which allows realizing the functionality of a GIS.
Geomatics - A scientific and technical direction that combines methods and means of integrating information technologies for the collection, processing and use of spatial data, including geoinformation technologies.
Geometric accuracy of the map - The degree to which the location of points on the map corresponds to their location in reality.

Geometric leveling - A method for determining elevations by sighting with a horizontal beam using a level and reading the height difference along the rails. Reading accuracy on rails I-2 mm (technical leveling) and up to 0.1 mm (high-precision leveling).
Geomorphological maps - Display the relief of the earth's surface, its origin, age, shape and size. A distinction is made between general geomorphological maps of broad content and private maps compiled according to individual features of the relief.
Geoportal - An electronic geographic resource hosted on a local network or the Internet, a website.
Georeferenced image (image) - An image (image) that has parameters for recalculation into the spatial coordinate system of the Earth.
Geospatial reference - The procedure for converting the coordinates of an object into the spatial coordinate system of the Earth.
Geospatial data - Digital data about spatial objects, including information about their location and properties (spatial and non-spatial attributes).
Geocentric coordinates - Values that determine the position of points in space in a coordinate system in which the origin of coordinates coincides with the center of mass of the Earth.
Geodetic surveys - A set of works carried out to study the topographic conditions of the construction area. We have been doing this work in our company for more than 10 years. http://tochno-rostov.ru/inzhenernye-izyskaniya/geodezicheskie-izyskaniya/
Global Navigation Satellite System (GNSS) - A system consisting of a constellation of navigation satellites, a monitoring and control service and user equipment that allows you to determine the location (coordinates) of the consumer's receiver antenna.
Global Positioning System (GPS) - GNSS developed in the USA.
Globe - Cartographic image on the surface of the ball, preserving the geometric similarity of the contours and the ratio of areas. There are: geographic globes that display the surface of the Earth, lunar - the surface of the Moon, celestial, etc.
GLONASS - GNSS developed in Russia
Geographic longitude - A dihedral angle enclosed between the plane of the meridian passing through a given point (observation point) and the plane of the initial meridian (Greenwich).
Geodesic longitude - Dihedral angle between the plane of the geodesic meridian of a given point and the plane of the initial geodesic meridian (GOST 22268-76).
General geographic maps - Display with the same detail the main natural and socio-economic objects (relief, vegetation, hydrography, settlements, borders, etc.). Large-scale general geographical maps include topographic maps.
General Earth Ellipsoid - An ellipsoid that best fits the surface of the geoid as a whole.
Geographic meridian - A line section of the surface of the globe by a plane drawn through any point on the earth's surface and the axis of rotation of the Earth. Prime meridian - the meridian from which the geographical longitude is counted; in international practice, the Greenwich Meridian is taken as the prime meridian.
Gravimetry - A branch of the science of measuring the quantities characterizing the Earth's gravitational field and using them to determine the shape of the Earth, study its general internal structure, the geological structure of its upper parts, solve some navigation problems, etc.
Geodetic binding - Integration (combination) of new geodetic data with previously created ones.
Geodetic signal - A structure in the form of a double pyramid 40-50 m high, which serves as a geodetic sign for points of the state geodetic network of high accuracy.
Geomagnetic latitude - The angular distance from the geomagnetic equator to the considered point on the earth's surface. It is counted along a great circle passing through a given point and geomagnetic poles.
General geographic map - A map that displays the totality of the main elements of the terrain.
Geodetic point - A point of a geodetic network fixed on the ground.
Geodetic center - An underground structure made of concrete, reinforced concrete, stone or metal, fixing the position of a geodetic point on the ground and designed for long-term preservation.

H

Hood - An accessory of a geodetic instrument designed to protect the telescope lens from direct sunlight.
Hillock - A hillock on a piece of land on the earth's surface of a domed shape. The relative height of the hillock does not exceed 100 m.
High-precision geodetic network - A network that provides the implementation of a coordinate system next in accuracy after the fundamental network, based on FAGS points. The main part of the methodology for creating the WGS is satellite determinations.
Higher geodesy - A section of geodesy that deals with determining the figure, size and gravitational field of the Earth. The tasks of higher geodesy also include the study of theories and methods of basic geodetic work , which serve to build a reference geodetic network and deliver data for solving scientific and practical problems of geodesy.
Heliotrope - Device, the main part of a flat mirror, which reflects the sun's rays from one geodetic point to another when triangulating.
Horizon - A curve that limits the part of the earth's surface available to the eye (visible horizon). The apparent horizon increases with the height of the observation point and is usually located below the true (in mathematics) horizon - a great circle along which the celestial sphere intersects with a plane perpendicular to the plumb line at the observation point.
Horizontal survey - A type of topographic survey , as a result of which a planned image of the area is created without a height characteristic of its relief.
Horizontal Angle - An angle in a horizontal plane corresponding to a dihedral angle between two vertical planes passing through a plumb line at the corner's vertex. Horizontal angles vary from 0° to 360°.
Hollow - A depression bounded by slopes of various steepness and shape, has considerable dimensions.
Hollow - An elongated depression with gentle slopes, sloping in one direction. The depth of the hollows is up to 10 m.
Hydrogeological maps - Display the conditions for the occurrence and distribution of groundwater; contain data on the quality and productivity of aquifers, the position of the ancient foundations of water systems, etc.
Hydroisobats - Isolines of the depths of the groundwater mirror from the earth's surface.
Hydroisohypses - Isolines of marks of the groundwater mirror relative to the conditional zero surface.
Hydroisopleths - Isolines of soil moisture at different depths at different times; points of the same water levels in different wells at different times.
Hydroisotherms - Isolines of water temperature in a given rock mass.
Hydrological maps - Display the distribution of water on the earth's surface, characterize the regime of water bodies and allow you to evaluate water resources.
Hydrostatic leveling - Determining the heights of points on the earth's surface relative to the starting point using communicating vessels with liquid. It is based on the fact that the free surface of the liquid in communicating vessels is at the same level. They are used for continuous study of deformations of engineering structures, high-precision determination of the height difference of points separated by wide water barriers, etc.
Hurstmonceaux - 15th century castle, modern location of the Greenwich Astronomical Observatory.
Hill - A small hill on a piece of land on the earth's surface, round or oval in shape with gentle (no more than 30 °) slopes and a slightly pronounced foot. Relative height is not more than 200 m.

I

Inverse geodetic problem - It consists in determining the geodetic coordinates of two points on the earth's ellipsoid of the length and directional angle of direction between these points.

Isolines - Lines of equal value of any value on a geographical map, vertical section or graph. Isolines give a characteristic of continuous phenomena in a certain period or point in time (for example, isobars, isobaths). They are used in mapping natural and socio-economic phenomena; can be used to obtain their quantitative characteristics and to analyze the correlations between them.
Interpolation - Recovery of a function on a given interval from its known values in a finite set of points belonging to this interval.
Ionospheric delay (with satellite definitions) - Change in the speed (delay) of propagation of electromagnetic radiation propagating from a satellite to a receiver when passing through the ionosphere.
Image overlap - In photogrammetry, the proportion of the area of an image (aerial image) that is overlapped by an adjacent image.
Initial geodetic dates - A set of values that determine the position of the reference ellipsoid adopted for processing the geodetic network of any country or group of countries, relative to the geoid, i.e., values that fix the position of the reference ellipsoid in the body of the Earth.
Integrated mapping - Multilateral mapping of natural and socio-economic phenomena on maps, taking into account their interrelations; is carried out by creating a series of interrelated thematic maps or their integral set (for example, an atlas).
Inch - Russian and English measure of length; 1 inch = 2.54 cm.

K

Krasovsky ellipsoid - Earth ellipsoid, determined from degree measurements in 1940 under the guidance of F. N. Krasovsky. Reference ellipsoid dimensions: semi-major axis (radius of equators, polar contraction 1:298.3.
Kremalera - A device for focusing the telescope of a geodetic instrument.
Kroki - A drawing of a piece of terrain, displaying its most important elements, made during visual survey.
Kurgan - A rounded hill with a distinct sole. Relative height - no more than 50 m. It is an anthropogenic form of relief.
Kipregel - A geodetic tool for drawing directions and determining distances and elevations in scale survey.

L

Landscape - A unit of physical and geographical zoning of a territory.
Landscape maps - Display the location of natural-territorial complexes of various ranks: facies, natural boundaries, groups of natural boundaries or localities (on large- and medium-scale maps), landscapes (on small-scale maps).
Limbo - A flat ring divided by strokes into equal parts of a circle (for example, degrees, minutes, etc.).
Line of sight - A line that determines the direction of the sighting axis of the geodetic instrument when pointing at a selected point.
Loxodrome (loxodrome) - A line on a sphere (or some other surface of revolution) that intersects all meridians at a constant angle K. On maps in the Mercator projection, loxodromies are shown as straight lines. Used by navigation and air navigation.
Land Cadastre - A systematized set of reliable information about the natural, economic and legal status of lands. Land cadastre data is used for taxation of landowners, registration of land transactions, pledge of land, etc. The land cadastre indicates the name of the land owner, describes the total area, location of plots, their configuration, composition of land, their quality, profitability, land price .
Land plot - As an object of land relations - a part of the earth's surface (including the soil layer), the boundaries of which are described and certified in the prescribed manner.

Land management - Measures to study the state of land, plan and organize the rational use of land and their protection, the formation of new and streamline existing land management facilities and the establishment of their boundaries on the ground, the organization of the rational use of land plots by citizens and legal entities for agricultural production, as well as the organization territories used by communities of indigenous peoples of the North, Siberia and the Far East of the Russian Federation and persons belonging to the indigenous peoples of the North, Siberia and the Far East of the Russian Federation to ensure their traditional way of life.
Laser rangefinder - A device for measuring distances using a laser beam.
Leveling - The operation of combining the vertical axis of the measuring instrument with a plumb line and (or) bringing the sighting axis of the telescope to a horizontal position.
Longitude - A coordinate that determines the position of a point on Earth in the West-East direction.
Leveling mark - A metal disc with a diameter of 8-10 cm with a hole in its center of about 2 mm, built into the wall of a stone structure (building, tower, bridge, etc.) from the outside and serving to fix a geodetic benchmark, the height of which above sea level is determined through leveling.
Least squares method - One of the error theory methods for estimating unknown quantities from measurement results containing random errors.
Level - A geodetic instrument designed to determine elevations.
Leveling network - A system of points on the earth's surface, the heights of which above sea level are determined by leveling and fixed on the ground with benchmarks.
Leveling - Determination of elevations between points on the earth's surface, and then the heights of points relative to some chosen point or above sea level. There are geometric, trigonometric and other types.
Leveling the route - Type of geodetic work , aimed at determining the elevations of points fixed on the axis of the route.
Lowland - A type of plain lying at an altitude of 0 to 200 m above sea level or below it.
Landmark - An immovable object (natural or artificial) or an element of relief that is clearly visible on the ground, helping to navigate the terrain, determine the direction when moving.
Latitude - One of the coordinates in a number of spherical coordinate systems that determines the position of points on the surface of the Earth, the Sun, planets and on the celestial sphere relative to the equator (ecliptic).
Level surface - At all its points, the potential of gravity has the same value. The level surface of the Earth's gravitational field coincides with the average water level of the World Ocean.
Level - Device for checking the horizontalness of lines and surfaces and measuring small angles of inclination. The main part is a glass ampoule filled with a light liquid (with the exception of a small volume of a "bubble").
Locator - A device for determining the location and depth of underground utilities through which current flows (for example, power cables, pipelines).
Light distance meter - A device for measuring distances by the time it takes for optical radiation (light) to pass the measured distance.
Latitude Service - Conducting research by astronomical institutions (more than 40 observatories of the world) on changes in the geographic latitude of their location, reflecting the change in the position of the poles on the Earth's surface (or the axis of rotation in the Earth's body). The Latitude Service is managed by the International Pole Traffic Service.

M

Map - Built in map projection; a reduced, generalized image of the surface of the Earth, the surface of another celestial body or extraterrestrial space, showing the objects located on them in a certain system of conventional signs.
Map legend - A set of conventional signs and explanations for the map.
Magnetic declination - The angular difference between the magnetic and true north (north), due to the fact that the magnetic north pole of the Earth is shifted relative to the true, geographical one.
Magnetic maps - Display with the help of isolines (isogon, isoclines, isodynes) the distribution of the geomagnetic field over the Earth's surface.
Map projection - Displaying the surface of an ellipsoid or sphere on a plane.
Mountain - An elevation on a piece of land on the earth's surface, domed or conical in shape, with slopes of considerable steepness. The relative height of the mountain is more than 200 m.
Mathematical cartography - A branch of cartography that studies the mathematical basis of maps.
Military topography - A branch of topography that studies the methods and means of obtaining information about the terrain in the interests of the combat activities of troops.
Mine surveying - The branch of mining science and technology, the subject of which is the study, on the basis of measurements and subsequent geometric constructions, of the structure of the deposit, the shape and size of the mineral bodies in the bowels, the placement of useful and harmful components in them, the properties of the host rocks, the spatial arrangement of workings, rock deformation processes and the earth's surface in connection with mining operations, as well as a reflection of the dynamics of the production process of a mining enterprise.
Menzula - Field drawing table, consisting of a tablet, a tripod and a stand fastening them.
Meridian magnetic - Projection of the geomagnetic field line on the Earth's surface. Magnetic meridians are complex curves converging at the north and south magnetic poles of the Earth.
Mechanical leveling - Determining the heights of points on the earth's surface relative to the starting point by automatically drawing the terrain profile and the measured distance .
Mission (with satellite definitions) - The procedure for setting parameters in the receiver of the conditions for observing satellites and modes of operation.
Mounting horizon - The level of production of construction and installation works.
Measuring tape - A measuring tool designed: for measuring extended linear objects (up to 100 m), and for marking rooms. When performing geodetic work, it is always used when measuring the height of the device, measurements of buildings, etc. The main element of the tape measure is a flexible tape calibrated in a metric or other measurement system.
Magnetic equator - The locus of points on the earth's surface at which the magnetic inclination is zero.
Measurement errors - Deviation of the result of geodetic measurements from the true (actual) value of the measured geodetic quantity.

N

Navigation satellite - A satellite that emits a radio signal containing navigation information, the reception of which is necessary to determine the location of the consumer's receiver.
Navigation satellite receiver - An apparatus consisting of an antenna, a radio receiver and a computer (processor) designed to receive and process navigation signals in order to obtain the information necessary for the consumer (spatio-temporal coordinates, direction and speed).
Nadir - The point of intersection of the celestial sphere with a plumb line extended down from the observation point. Nadir is opposite the zenith.
Nomenclature of topographic maps and plans - A system for marking and designating individual sheets of a multi-sheet map.
Normal - Perpendicular to the surface of the given point.

Normal planes and sections - All planes passing through the normal are called normal planes, and their section of the ellipsoid surface is called a normal section.
North (point of north) - The point of intersection of the mathematical horizon with the celestial meridian, closest to the North Pole of the World.
North Pole - The intersection point of the Earth's axis of rotation with its surface in the Northern Hemisphere.

O

Onboard ephemeris (navigation satellites) - Information about the location of satellites in orbit, transmitted as part of the measurement information.
Outline - A schematic drawing of an area that displays the features needed to draw up a topographic plan or profile
Ordinar - Zero footstock at water metering posts, fixing the average long-term water level in reservoirs. Level fluctuations are counted above and below the ordinary with an accuracy of 1 cm.
Orientation of lines - Determination of their directions relative to other lines.
Orthomosaic - The process of brightness alignment and merging ("stitching") of several orthorectified images (images) into one continuous image (image) with a predetermined pictorial quality.
Ortho-transformation (ortho-correction) of an image (image) - Mathematically rigorous transformation of the original image (image) into an orthogonal projection and elimination of distortions caused by terrain, shooting conditions and camera type.

P

Paleogeographic maps - Display the physical and geographical conditions of the geological past (the distribution of land, sea and river network, the nature of the relief of the continents, climatic features, etc.).
Palette - A transparent plate with a grid of lines applied to it (less often - points), designed to calculate areas on plans and maps, read coordinates, etc.
Pantometer - Goniometric geodetic instrument used in surveying forests and peat bogs.
Parallax - A visible change in the position of an object (body) due to the movement of the observer's eye.
Parallel - A line of section of the surface of the globe by a plane parallel to the plane of the equator. All points on this line have the same latitude.
Primary level surface - A surface that at each point is perpendicular to the direction of a plumb line and has a constant gravity potential.
Plumb line - A straight line coinciding with the direction of gravity at a given point.
Perspective aerial photography - Photographing the terrain with an aerial camera, the optical axis of which is deviated from the vertical by a certain constant angle.
Picket - A point on the ground (indicated by a sign), which serves as a guide for setting the rail when leveling and for fixing the route on the ground. Fixes the specified interval.
Plan - 1) A drawing depicting in conventional signs on a plane (on a scale of 1:10,000 and larger) part of the earth's surface (topographic plan) and built without taking into account the curvature of the Earth. 2) A horizontal section or top view of any structure or object. 3) Same as horizontal projection.
Planimeter - A mechanical or electronic device for measuring the areas of objects according to plans and maps.
Planned aerial photography - Photographing the area with the position of the optical axis of the aerial camera close to vertical.

Planned staking basis - Geodetic construction at the construction site, providing mutual coordination of all design elements of the complex and serving to obtain initial data for stakeout.
Plateau - A vast area of the earth's surface, which is a mountain plain, characterized by significant erosional dissection.
Polygonometric point - A geodetic point, the coordinates of which are determined by the polygonometry method, and the position on the ground is indicated by metal pillars or concrete monoliths.
Polygonometry - A method of constructing a geodetic network in the form of a broken line in which all sides and angles are measured.
Polyconic projections - Cartographic projections, the parallels of which are the arcs of eccentric circles, and the meridians are curves symmetrical about the average rectilinear meridian. Used for world maps.
Polar coordinates - A system of plane coordinates formed by a directed direct ray OX, called the polar axis. Most often, the north axis of a meridian is taken as the polar axis. The origin of coordinates - point O - is called the pole of the system.
Post-processing (satellite observations) - Final processing of data in office conditions in order to obtain the coordinates of points.
Project line - A line that determines the position of structures in terms of and in height.
Profile - A vertical section, a section of any part of the earth's surface, earth's crust, hydrosphere or atmosphere along a given line.
Peak - The top of a mountain or the highest part of a mountain range or spur.
Plotter (plotter, auto-coordinator) - a display device designed to output data in graphical form onto paper, plastic, photosensitive material or other media by drawing, engraving, photographic recording or otherwise.
Point height (elevation) - The distance measured in the direction of the plumb line from the given point to the reference surface.
Plain - A section of the earth's surface characterized by slight slopes and fluctuations in elevation. Together, the plains occupy most of the Earth's surface and are the most important element of the relief. The greatest plain in the world is the Amazonian (over 5 million km2).
Projection Coordinate System - A two-dimensional coordinate system resulting from a mapping project.
PZ-90 system - Russian system of geodetic parameters of the Earth in 1990, used in GLONASS, which includes a system of geocentric coordinates.
Photogrammetry - Determining the shapes, sizes, and positions of objects from their photographic images.
Photomaps - Combine a planned photographic image of the area with its cartographic image (for example, the relief is shown by contour lines, etc.).
Phototopography - A section of topography that studies methods for creating topographic maps based on ground photography.
Phototriangulation - A method for determining the coordinates of terrain points from photographs.
Parts of the world - Land regions of the Earth, including the continents or large parts of them, together with nearby islands. Usually 6 parts of the world are distinguished Europe, Asia (one continent Eurasia), Africa, Australia, America (two continents - South America and North America), Antarctica; sometimes Oceania. The division of land into parts of the world has developed historically and differs from the division into continents, as well as into the Old and New Worlds.

R

Radian - A unit of measure for plane angles, which is a central angle based on an arc whose length is equal to the radius of this circle. It is usually used in theoretical calculations.

Radio geodetic systems - A complex of radio engineering devices for determining the coordinates of photographing points during aerial photography by measuring distances from an aircraft to points on the earth's surface with known geographical coordinates using radio rangefinders.
Radar survey - Obtaining images of the terrain using radar equipment installed on aircraft. It can be carried out in difficult meteorological conditions and at any time of the day to study objects (including those covered with snow, vegetation, loose sediments, etc.).
Raster image (raster), raster data - An image in the form of an array of pixels obtained as a result of shooting with digital frame or scanner cameras installed on air or space media, or as a result of scanning an image from a photographic film (image) or paper using a scanning devices. A raster can also be obtained by converting (rasterizing) vector graphic data into a raster image using special software tools.
Rail - A wooden beam 3-4 m high with divisions of 1-5 cm, installed vertically at the observed points during leveling and topographic survey.
Rail heel - The rail base, designed to be mounted on a benchmark, shoe or crutch.
Reconnaissance - Inspection and survey of the area in order to select the position of geodetic reference points to justify topographic surveys and moves.
Relief - A set of irregularities of the land, the bottom of the oceans and seas, various in shape and size.
Remote sensing - The process of obtaining information by remote methods about the surface of the Earth and other space bodies, objects located on it or in its bowels.
Remote methods - Non-contact methods for studying the Earth's surface, hydrosphere, lithosphere, atmosphere and space bodies (remote sensing, airborne geophysical methods, sonar surveys of the bottom of water areas).
Reference ellipsoid - An ellipsoid that serves as an auxiliary mathematical surface, to which the results of geodetic measurements on the earth's surface lead. In the Russian Federation, the Krasovsky ellipsoid is adopted.
Refraction - Various types and manifestations of refractive electromagnetic waves, due to the curvature of the trajectory of the propagation of these waves and accompanying all kinds of geodetic measurements.
Rumb - The angle between the meridian and the given direction, counted from the meridian in both directions from 00 to 900; in maritime navigation, a measure of the angle of the circumference of the horizon divided by 32 rhumbas (in meteorology by 16).
Relief section - The difference in heights between two successive horizontals on a topographic map or plan.
Route - The axis of the designed linear structure, indicated on the ground or plotted on the map.
Rectangular coordinates - A planar coordinate system formed by two mutually perpendicular straight lines, called the x and y coordinate axes. The point of their intersection is called the origin or zero of the coordinate system. The abscissa axis is OX, the ordinate axis is OY.
Reference geodetic network - A system of points fixed on the ground, the planned position and height of which are determined in a single coordinate system based on geodetic measurements; these points serve as reference points for geodetic and topographic surveys.
Reference boundary network - the Special Purpose Geodetic Network (GSSN), which is created for geodetic support of the state land cadastre, land monitoring, land management and other measures to manage the country's land fund.
Red lines - Lines that indicate existing, planned (changeable, newly formed) boundaries of common areas, boundaries of land plots on which engineering and technical support networks, power lines, communication lines, pipelines, roads, railway lanes and other similar structures.
Rangefinder - A device designed to determine the distance from the observer to the object.
Resection - A method of determining the coordinates of a point by measuring parameters on it or on starting points with known coordinates.
Ridge - A mountain range that extends in one direction. The longest range is the Andes (8500 km).

Relative height - Excess, the difference in absolute heights of any point on the earth's surface relative to another point.

S

Surveying (land surveying) - A complex of urban planning and land management works to establish, restore and fix on the ground the boundaries of a land plot (as a property), determine its location and area. We are actively engaged in this type of work. More detailed information on the service can be obtained here http://tochno-rostov.ru/kadastrovye-raboty/mezhevanie/ .
Scale survey - A type of topographic survey carried out using a kipregel and a scale; in the process of scale surveying, a plan with contour lines is created graphically directly when surveying the terrain.
Stakeout stationing - Stakeout of the station route on a circular curve and assign curve radii.
System time scale (SWM) - The time scale of the highest accuracy, designed to synchronize the operation of all GNSS segments, is formed and maintained by the most stable time standards located in control and management systems and associated with national frequency standards.
Satellite (geodetic) definitions - Determination of coordinates of points or increments of coordinates between points, based on the processing of measurement information coming from GNSS satellites.
Satellite geodetic networks - Geodetic networks created by methods of satellite definitions.
Saddle - A drop in the watershed between two hills.
Scale - The ratio of the length of a line in a drawing, plan or map to the length of the corresponding line in kind.
Spatial data - Digital data about spatial objects, including information about their location, shape and properties, presented in a coordinate-time system.
Stereopair - Two images of the same area, belonging to photographs obtained at different positions of the projection center.
Stereotopographic survey (stereophotogrammetric survey) - A method of creating an original topographic map based on the processing of photographic images of the area using stereophotogrammetry methods. As a result of stereotopographic survey, the plan and elevation positions of terrain points are determined, aerial photographs are deciphered, the relief is stereoscopically drawn, and the original map is compiled.
Stereophotogrammetric devices - Optical-mechanical and electronic devices, supplemented in some cases by computers and automation equipment; allow to determine the size, shape and position (coordinates) of the objects depicted on them from stereoscopic images of the terrain (stereoparas), as well as to draw topographic plans and maps.
Stereophotogrammetry - A section of photogrammetry that studies methods for measuring three-dimensional forms (for example, terrain) from a stereo pair of photographs, based on the use of the stereoscopic effect and the measurement of a three-dimensional terrain model with special stereometric devices.
Stratoisohypses - Isolines of the absolute or relative marks of the surface of any geological bodies (formation, intrusive body, etc.). Used on underground relief maps or structural maps.
Shooting the situation - Geodetic measurements on the ground for subsequent drawing on the plan of the situation (contours and terrain objects).
Scale accuracy - Limit - a segment of 0.1 mm, graphic - 0.2 mm.
Stroke length - The distance between the start and end points of the traverse, obtained as the sum of the lengths of all sides of the traverse.
State Geodetic Network - A system of points fixed on the ground, the position of which is determined in a single system of coordinates and heights.
Space photography - Shooting (photographic, television, etc.) of the Earth, celestial bodies and cosmic phenomena with equipment located outside the Earth's atmosphere (on artificial Earth satellites, spacecraft, etc.) and giving images in various regions of the electromagnetic spectrum.

Slope markers - Same as berghatches .
Slope - An indicator of the steepness of the slope; the ratio of terrain elevation to the horizontal extent over which it is observed (for example, a slope of 0.0I5 corresponds to a rise of I5 m over I 000 m of distance).
South (south point) - The point of intersection of the mathematical horizon with the celestial meridian, closest to the South Pole of the world. Denoted Yu or S.

T

Total station - Geodetic instrument for measuring distances, horizontal and vertical angles. Automatic total stations allow you to determine angles and distances without calculations.
Tacheometric survey - A type of topographic survey in which horizontal and vertical angles are measured by the circles of the tacheometer, and the distances to objects are measured by its range finder. It is used to create a site plan with contour lines for engineering surveys, geological, hydrological and other studies.
Theodolite - A geodetic instrument designed to measure horizontal and vertical angles or zenith distances.
Theodolite survey - Horizontal geodetic survey of the area, performed to obtain a contour plan of the area (without a height characteristic of the relief) using a theodolite.
Topographic survey - A set of works on the creation of an original topographic map using aerial phototopography methods or for small areas of terrain by ground surveys (scale, tacheometric, etc.). The Tochno company has been practicing this type of work for many years: http://tochno-rostov.ru/topograficheskaya-semka/
Topographic symbols - Symbolic graphic designations used on topographic maps to depict terrain objects and their qualitative and quantitative characteristics. There are scale (areal and linear), off-scale and explanatory signs.
Target - A vertical plane passing through two points.
Tablet - 1) Part of the scale, a square wooden board (side size from 40 to 70 cm), on which drawing paper is pasted. 2) A board or folder on which the compass and paper are fixed during eye survey.
Tape - A geodetic device designed for direct measurement of distances on the ground.
Topographic surveys - Field and cameral work in order to draw up plans and maps of the earth's surface.
Triangulation - A method of determining the position of geodetic points by constructing systems of adjacent triangles on the ground, in which the length of one side (according to the basis) and angles are measured, and the lengths of other sides are obtained trigonometrically. The main method for creating a reference geodetic network and degree measurements.
Trigonometric point (triangulation point) - Geodetic point, the coordinates of which are obtained by triangulation; position on the ground is indicated by a wooden or metal structure in the form of a pyramid.
Trigonometric (geodesic) leveling - A method for determining elevations by the measured angle of inclination of the line of sight from one point to another and the distance between these points. It is used in topographic surveys and other works.
Trilateration - A method for determining the position of geodetic points by constructing systems of adjacent triangles on the ground, the coordinates of the vertices and angles of which are determined trigonometrically, and the lengths of the sides are determined using rangefinders.
Tropospheric delay (with satellite definitions) - Change in the speed (delay) of the propagation of electromagnetic radiation propagating from the satellite to the receiver when passing through the troposphere (non-ionized part of the atmosphere).
Tasks of engineering geodesy - Determined by the type and composition of geodetic measurements carried out for the purposes of construction production.

The State Leveling Network is a unified system of heights throughout the country, it is the high-altitude basis of all topographic surveys and engineering and geodetic work performed to meet the needs of the economy, science and defense of the country.
The Fundamental Astronomical Geodetic Network (FAGS) is a network that provides the highest level of accuracy of the global geocentric coordinate system on the territory of Russia. It is characterized by errors in determining the coordinates of points relative to the center of mass of the Earth, not exceeding 15 cm, and errors in the relative position, not exceeding 2 cm. The average distances between points are km. A significant part of the method of creating this network is satellite determinations
Tripod - A device, most often in the form of a folding tripod or clamp, for rigid fixation of instruments.
Topographic map - a general geographical map on a scale from 1:1 to 1:10,000, which conveys with great accuracy and detail the main natural and socio-economic objects (relief, vegetation, hydrography, settlements, road network, etc.) and allows you to determine both the planned and vertical position of the points. It is built on a rigid geodetic basis in a stable system of symbols.
The steepness of the slope (slope) - The angle formed by the direction of the slope with the horizontal plane and expressed in angular measures or slopes.

U

Universal processing program (satellite observations) - A program that allows you to post-process satellite definitions made by receivers of various GNSS systems, as well as measurements made by other systems (for example, satellite laser ranging systems, long-baseline radio interferometry systems) (Bernes, GYPSY, GAMIT, etc. .).
Universal tool - A portable goniometric tool for solving many problems of practical astronomy and geodesy, in particular for measuring the coordinates (heights and azimuths) of celestial bodies and terrestrial landmarks.
Urban geodetic network - Designed to provide practical tasks: - topographic survey and updating city plans of all scales; - land management, land surveying, land inventory; - topographic and geodetic surveys in the urban area; - engineering and geodetic preparation of construction objects; - geodetic study of local geodynamic natural and man-made phenomena in the city;

V

Vernier - A device with which the fractions of divisions of the main scale of the limb are counted in geodetic instruments. The action of the vernier is based on the ability of the eye to confidently match 2 strokes when one of them is a continuation of the other and their ends coincide.
Vertical - A large circle of the celestial sphere, passing through the zenith and nadir. The vertical whose plane is perpendicular to the meridian is called the first vertical. The intersection of the first vertical with the celestial horizon gives a point of west and east.
Vertical angle - Angle in the vertical plane (tilt angle, zenith distance, etc.).
Vector - A straight line segment with a certain direction, starting from the starting point and arriving at the end point. Characterized by numerical value and direction.
Vector image - Digital representation of point, line and polygonal spatial objects as a set of coordinate pairs.
Viewfinder - A device, a device for visually aiming a goniometer, rangefinder or observation device at a specific point in space.
Valley - An elongated depression with a slope in one direction, having slopes of various steepness and shape.

W

Watershed - A line on the earth's surface that runs along the ridge along the ridge and connects its most elevated points.
WGS-84 - The 1984 World Geodetic System of Earth Parameters used in GPS, which includes the geocentric coordinate system.

Z
Zenith distance - The vertical angle measured from the direction of a plumb line (astronomical zenith distance *)* or from the normal to the ellipsoid (geodetic zenith distance) from 0 ° to 180 °.

GEODESY TESTS

1. The science that studies the shape, dimensions of the globe or individual sections of its surface by measurements
1) topography;
2) cartography;
3) geodesy;
4) geology;

2. The surface formed as a conditional continuation of the world ocean under the continents is:
1) physical surface;
2) main level surface;
3) horizontal surface;
4) the surface of the ellipsoid.

3. The figure of the Earth, formed by a level surface coinciding with the surface of the World Ocean in a state of complete rest and equilibrium, is continued under the continents in accordance with this:
1) in-terrestrial ellipsoid;
2) geoid;
3) reference ellipsoid;
4) the globe.

4. Approximation of the shape of the earth's surface (geoid) to an ellipsoid of revolution, which is used for the needs of geodesy on a certain part of the earth's surface:
1) quasi-geoid;
2) rivneva surface;
3) reference ellipsoid;
4) earth ellipsoid.

5. The dimensions of the earth's ellipsoid characterize:
1) lengths of parallels and meridians;
2) latitude and longitude;
3) the average radius of the Earth;
4) the length of the semi-major axis and the polar compression.

6. The lines of the section of the surface of the ellipsoid by planes that pass through the axis of rotation of the Earth are:
1) meridians;
2) parallels;
3) normals;

4) plumb lines.

7. The lines of the section of the surface of the ellipsoid by planes that are perpendicular to the axis of rotation of the Earth are:
1) meridians;
2) parallels;
3) normals;
4) plumb lines.

8. Three quantities, two of which characterize the planned position, and the third is the height of a point above the surface of the earth's ellipsoid - these are:
one). Cartesian coordinates;
2) topocentric coordinates;
3) geodetic coordinates;
4) geocentric coordinates.

9. The angle formed by the normal to the surface of the earth's ellipsoid at a given point and the plane of its equator (up or down from the equator) is:
1) geodetic longitude;
2) geodetic latitude;
3) astronomical longitude;
4) astronomical latitude.

10. The dihedral angle between the planes of the geodesic meridian of a given point and the initial geodesic meridian (to the right or left of the zero meridian) is:
1) geodetic longitude;
2) geodetic latitude;
3) astronomical longitude;
4) astronomical latitude.

11. The height of a point above the surface of the earth's ellipsoid is:
one). geodetic height;
2) orthometric height;
3) dynamic height;
4) normal height.

12. The height of the point, determined relative to the main level surface, is:
1) relative height;
2) absolute height;
3) the applicant point;
4) geodetic height.

13. In Ukraine, absolute heights are determined in:
1) Dnieper height system
2) Baltic height system
3) Black Sea height system
4) Azov system of heights.

14. The height difference of two points is:
1) excess;
2) applique increments;
3) abscissa increments;
4) increments of ordinates.

15. Leveling is understood as field work, as a result of which:
1) elevation between individual points;
2) rectangular coordinates of points;
3) polar coordinates of points;
4) geodetic coordinates of points.

16. a miniature image of a part of the earth's surface, created without taking into account the curvature of the earth, is:
1) map of the area;
2) plan of the area;
3) terrain profile;
4) outline of the terrain.

17. Reducing the generalized image on the plane of the entire or a significant part of the earth's surface, compiled in the accepted cartographic projection, taking into account the curvature of the Earth, is:
1) map of the area;
2) plan of the area;
3) terrain profile;
4) outline of the terrain.

18. Images on the plane of the vertical section of the terrain surface in a given direction are:
1) map of the area;
2) plan of the area;
3) terrain profile;
4) outline of the terrain.

19. The totality of the contours and objects of the area indicated on the plan is:
1) relief;
2) situation;
3) profile;
4) outline.

20. Irregularities of the earth's surface of natural origin are:
1) terrain;
2) the situation of the area;
3) terrain profile;
4) outline of the terrain.

21. In the case of a contour (horizontal) survey on a map or plan, the following is displayed:
1) terrain;
2) the situation of the area;
3) terrain profile;
4) relief and situation of the area.

22. In the case of a topographic survey, the map or plan shows:
one). the contours of the object;
2) boundaries of adjacent sections;
3) terrain profile;
4) relief and situation of the area.

23. In the case of a cadastral survey, the plan shows:

1) terrain;
2) terrain profile;
3) the relief and situation of the area;
4) the contours of the object, the situation and the boundaries of adjacent areas. +

24. The main cartographic projection for topographic and geodetic works in Ukraine is:
1) Mercator projection;
2) projection of Zoldner coordinates;
3) Gauss-Kruger projection;
4) projection of Sanson.

25. In the coordinate system built on the basis of the Gauss-Kruger projection, the x-axis is taken as:
1) axial meridian of the zone;
2) the meridian of a given point;
3) Greenwich meridian;
4) equator.

26. In the coordinate system built on the basis of the Gauss-Kruger projection, the y-axis is taken as:
1) axial meridian of the zone;
2) the meridian of a given point;
3) Greenwich meridian;
4) equator.

27. In the coordinate system built on the basis of the Gauss-Kruger projection, the ordinate of the point is y \u003d 6520000 m, therefore this point is in the coordinate zone number:
1) 6;
2) 5;
3) 2;
4) 52)

28. In the coordinate system built on the basis of the Gauss-Kruger projection, the ordinate of the point is y \u003d 5420000 m, therefore this point is in the coordinate zone number:
1) 15;
2) 4;
3) 2;
4) 42

29. The axial meridian on the topographic map is the same or parallel:
1) with horizontal lines of the kilometer grid
2) with vertical lines of the kilometer grid +
3) with horizontal lines of the inner frame of the card;
4) with vertical lines of the inner frame of the card.

30. The geographical coordinates of a point are determined by:
1) abscissa and ordinate;
2) latitude and longitude;
3) meridians and parallels;
4) angles and lengths of lines.

31. Rectangular geodetic coordinates of a point are determined by:

1) abscissa and ordinate;
2). latitude and longitude;
3) meridians and parallels;
4) angles and lengths of lines.

32. The origin of the coordinates in the Gauss-Kruger projection is taken :
one). intersection point of the Greenwich meridian and the equator line;
2) the point of intersection of the geographic meridian and the line of the equator;
3) the point of intersection of the projections of the axial meridian of the given zone and the line of the equator;
4) the point of intersection of the magnetic meridian and the line of the equator.

33. Geodesy studies?
1) Earth's surface.
2) The structure of the earth's crust.
3) Vegetation.
4) The surface of the seas and oceans.

34 . **Does the earth have a shape?**
1) Shara.
2) Spheres.
3) Ellipsoid.
4) Ellipsoid of revolution.

35. Conditional image on a topographical plan?
1) Vertical section of the terrain.
2) Geology.
3) Location.
4) Countries of the world.

36. Influence of curvature of the Earth's surface on measurements of lengths and heights.
Is this influence negligible over an area with a radius?
1)10km.
2)100km.
3)200km.
4)150km.

37. Coordinate system in geodesy on plans?
1) Polar
2) Rectangular.
3) Round.
4) Geographic coordinates.

38 . **Is the cartographic projection accepted in Uzbekistan?**
1) Lomonosov.
2) Ulugbek.
3) Gauss-Kruger.
4) Laplace.

39. Topographic map is it?
1) Graph.
2) Conditional image of the earth's surface.
3) Drawing.

4) Profile.

40. Scale accuracy 1: 500?
1)1meter.
2)0.5m.
3)0.05m.
4) 0.1m.

41. Orientation of lines means the direction is relative?
1) Countries of the world.
2) Equator.
3) Meridian.
4) South Pole.

42. Does the nomenclature of a topographic map define it?
1) The coordinate system.
2) Height system.
3) Scale.
4) Countries of the world.

43. Are there conventional signs of a topographic map?
1) Outline.
2) Dimensional.
3) Dimensionless.
4) Colored.

44. Is the relief depicted?
1) Hills.
2) Horizontals.
3) Slopes.
4) Lowlands.

45. Does the mortgage schedule reflect?
1) Mountain.
2) Hollow.
3) The steepness of the slope in meters.
4) The steepness of the slope in degrees.

46. Does the inner frame of the topographic map look like?
1) Rectangle.
2) Trapeze.
3) Square.
4) Stripes.

47. Are the western and eastern sides of a sheet of a topographic map segments?
1) Meridians.
2) Parallels.
3) Squares.
4) Rectangles.

48. Are the northern and southern sides of the topographic map segments?
1) Parallels.
2) Meridians.
3) Squares.

4) Rectangles.

49. What is indicated on the horizontal lines of the coordinate grid?
1) Ordinates
2) Abscissa.
3) Absolute marks.
4) The height of the relief.

50. What is indicated on the vertical lines of the coordinate grid?
1) Ordinates.
2) Abscissa.
3) Absolute marks.
4) The height of the relief.

51.Longitude and latitude have values in?
1) Degrees.
2) Meters.
3) Kilometers .
4)In whole numbers of kilometers.

52. Do abscissas and ordinates matter in?
1) Degrees.
2) Kilometers and meters.
3) Absolute marks.
4) Relative marks.

53. Is the scale map the basis of the nomenclature of topographic maps?
1)1 : 1 000 000 .
2)1 : 2,000,000.
3)1 : 10,000,000
4)1 : 10,000.

54. The size of the map sheet frame is 1:1,000,000 in longitude and latitude?
1)6 to 4 degrees .
2)4 to 6 degrees.
3)6 by 6 degrees.
4) 10 to 10 degrees.

55. A card sheet 1: 1,000,000 is divided into sheets 1: 100,000 in quantity?
1)100.
2)144.
3)150.
4)200.

56. A card sheet 1: 100,000 is divided into sheets 1: 50,000 in the amount of e?
1)10.
2)4.
3)20.
4)100.

57. Is a map sheet 1: 50,000 divided into sheets 1: 25,000 in quantity?
1)10.
2)4.
3)20.

4)100.

58. Is a map sheet 1:25,000 divided into sheets 1:10,000 in quantity?
1)10.
2)4.
3)20.
4)100.

59. What is the nomenclature of a map sheet 1: 1,000,000?
1) M-41.
2) M-41-60.
3) M-41-60-A.
4) M-41-60-A-g

60. What is the nomenclature of the map sheet 1,100,000?
1) M-41-144.
2) M-41-60-A.
3) M-41-60-A-g
4) M-41-60-A-g-4

61. What is the nomenclature of a map sheet 1: 50,000?
1) M-41-60.
2) M-41-60-A.
3) M-41-60-A-g
4) M-41-60-A-g-4

62. What is the nomenclature of a map sheet 1: 25,000?
1) M-41-60.
2) M-41-60-A.
3) M-41-60-A-g.
4) M-41-60-A-g-4

63. What is the nomenclature of a map sheet 1: 10,000?
1) M-41-60.
2) M-41-60-A.
3) M-41-60-A-g.
4) M-41-60-A-g-4.

64. Is it indicated in the corners of the frame of the topographic map?
1) Latitude and longitude.
2) Distance.
3) Angle.
4) Azimuth.

65. How to determine the area on the map?
1) By marks.
2) Square palette.
3) According to the directional angle.
4) By compass.

66. Is it possible to build a profile on the map?
1) horizontally.
2) vertically.
3) By coordinates.

4) In the corners.

67. In the field of view of the theodolite spotting scope, do we see?
1) Cylindrical level.
2) Round level.
3) A grid of threads.
4) Reading device of corners.

68. The main condition of the level?
1) Collimation error.
2) The place of zero is not equal to zero.
3) The sighting axis is parallel to the axis of the cylindrical level -1.
4) The sighting axis is parallel to the axis of the circular level.

69. Technical leveling performed?
1) Roulette.
2) Rail with a level.
3) Plumb.
4) Level type H3.

70. Are there theodolites and total stations?
1) Accurate and high precision.
2)Great precision.
3) Self-aligning.
4) Low precision.

71. Should the condition be observed in theodolite?
1) Perpendicularity of the sighting axis to the axis of rotation of the telescope
2) Straightness of the sighting axis.
3) Parallelism of the sighting axis to the level axis.
4) Equality of the lengths of the sight lines.

72. How to measure horizontal angles?
1) Receptions and repetitions.
2) Pointing rangefinding threads at the target
3) Way of alignments.
4) Method of perpendiculars.

73. The main errors in measuring angles arise due to?
1) Inaccurate centering.
2) Solar radiation.
3) Weak wind.
4) Cool weather.

74. Does it affect the accuracy of vertical angle measurement?
1) Collimation error.
2) Inequality of supports.
3) Unknown value of zero place .
4) Different length of tripod legs.

75. Instruments for measuring lengths include ?
1) Rangefinders and tape measures .
2) Levels.
3) Bussoli.

4) Goniometers.

76. Indirect measurement of lines?
1) Roulette.
2) Reiki.
3) compass.
4) Determination of impregnable distance .

77. What type of rangefinder is available in the scanner and total station?
1) Thread.
2) Scale.
3) Laser.
4) Differential.

78. State geodetic network is it?
1) Network 1 - 4 classes.
2) Network 5-10 class.
3) Network 10-15 class.
4) Network 15-20 class.

79. Devices for setting directions and planes?
1) Roulettes.
2) Reiki.
3) Theodolites and levels.
4) Tripods.

80. Centers and external signs of the geodetic network?
1) Geodetic signal.
2) Geodetic level.
3) Reverse plumb.
4) Railroad switch.

81. Methods for the development of geodetic networks?
1) Triangulation method.
2) Method of parallels.
3) Method of sighting.
4) Eye method.

82. Geodetic networks of condensation?
1) Leveling 1 class.
2) Triangulation of the 1st class.
3) Astronomical network.
4) Theodolite passages.

83. State leveling network?
1) Leveling move.
2) Theodolite traverse.
3) Scale shooting.
4) Leveling network I-IV class.

84. Leveling methods?
1) Geometric.
2) Astronomical.
3) Lunar.

4) Sunny.

85. Do they refer to geodetic networks?
1) Leveling network.
2) Northern network.
3) Southern network.
4) Western network.

86. What is measured in a theodolite traverse?
1) Measure the angles and lengths of lines.
2) Measure excesses.
3) Measure vertical angles.
4 Calculate excesses.

87. What is measured in the leveling course?
1) Measure horizontal angles.
2) Measure excesses.
3) Measure directions.
4) Measure the true azimuth.

88. What receivers are used in satellite navigation?
1) Satellite receivers.
2) Solar receivers.
3) Lunar receivers.
4) Astronomical azimuths.

89. Is the tracing of linear structures on the ground carried out?
1) Compass.
2) Protractor.
3) Theodolite.
4) Eyepiece.

90. The breakdown of pickets and cross-sections starts from?
1) The beginning of the route.
2) Curve vertices.
3) The center of the radius of a circular curve.
4) The water's edge in the river.

91. Do you calculate the rounding elements?
1) By radius and angle of rotation.
2) Along the length of the track.
3) On the slope of the track.
4) According to the guiding bias.

92. Leveling of the route and cross-sections is carried out?
1) Level.
2) Theodolite.
3) compass.
4) tripod.

93. What determines the slope of the track?
1) On the type and class of the track.
2) From the value of the radius of the circular curve.
3) On the length of the curve.

4) From picketing.

94. Type of geodetic survey?
1) Tacheometric.
2) Direct serif.
3) International.
4) Civil.

95. Horizontal survey in progress?
1) Theodolite.
2) compass.
3) Barometer.
4) Level.

96. Tacheometric survey in progress?
1) Total station.
2) Theodolite.
3) Level.
4) Barometer.

97. Methods of geodetic breakdowns?
1) Alignments and perpendiculars.
2) Laser.
3) Rangefinder.
4) Vertical.

98. Geodetic preparation of project stakeout?
1) horizontally.
2) vertically.
3) According to design drawings.
4) As directed by the boss.

99. Removal of design points in the plan?
1) Theodolite.
2) tripod.
3) Lifting screws.
4) Level.

100. Removal of design marks in height?
1) tripod.
2) Level.
3) Lens.
4) Theodolite.

101. Ways of detailed breakdown of the curve?
1) By constructing given angles and lines.
2) By building a given height.
3) Building a vertical.
4) Building a horizontal line.

102. Do you control the installation of columns vertically?
1) Theodolite.
2) Curvimeter.
3) Diopter ring.

4) Lifting screws.

103. Orient the line means?
1) Determine its position relative to the direction taken for initial.
2) Find the length of its horizontal projection.
3) Determine the height of its start and end points.
4) Put on a plan or map a horizontal projection of the line.

104. The length of the segment on the plan 1: 2000 is 15.85 cm. In this case, on the ground, its length is?
1) 31.7m.
2) 317m.
3) 3170m.
4) 3.17m.

105. Coordinates of a point in geodesy are called?
1) Distance from the origin to the given point.
2) The length of the projection of the line on the coordinate axes.
3) Angular and linear values that determine the position of a point on surface of the earth or in space.
4) The position of the point on the coordinate plane.

106. Geodetic angular measurements on the ground are made using?
1) Protractor.
2) Theodolite.
3) Spirit level.
4) Level.

107. Readings to the rear point (A) are: on the black side of the rail 1125, on the red side 5810; readings to the front point (B) are: on the black side of the rail 1553, on the red side 6240. In this case, the average excess will be equal to?
1) -430mm.
2)-428mm.
3) -4885mm.
4) -429mm.

108. The sighting axis of the telescope is called?
1) A line passing through the collimator sight and sighting target.
2) The horizontal axis of rotation of the theodolite telescope.
3) A line passing through the center of the horizontal limb and the sighting target.
4) A line passing through the center of the reticle and the optical center of the lens.

109. What is called measurement error?
1) Deviation of the measurement result from the true value of the measured quantities.
2) Error occurring when measuring the horizontal angle.
3) An error that must be taken into account in the mathematical processing of the results of field measurements.
4) An error caused by the non-perpendicularity of the vertical and horizontal axes of the theodolite.

110. The sum of the measured angles of a closed pentagonal theodolite traverse is $539^{o}58'$. Under these conditions, the angular discrepancy is?
1) $0^{o}01'$
2) $0^{o}03'$

3) 0º02′ .
4) 0º01′

111. Geodetic construction in the form of a broken line is called?
1) Geographic move.
2) Topographic move.
3) Engineering move.
4) Geodetic course.

112. Maps or plan, as well as obtaining topographic information in another form is called?
1. Topographic survey.
2. Field work.
3. Photographic shooting.
4. Office work.