Optimal Land Measuring Work Package for the Archaeological Research in Gorny Altai and Creation of the Specialized GIS (Geographic Information System)*

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1. INTRODUCTION

Archaeological site excavation entails partial or full destruction of a site. The final goal of any archaeological excavation is to create archaeological data source. The reliability of archaeological results is usually viewed based on the methods of data collection and on the ways, which this information is treated. These methods include verification and validity evaluation of the archaeological source methods. The foundation of any of the archaeological sites is characterized by a combination of factors, i.e. researchers' observations, recorded in a specific way. Modern methods of the field research presuppose the result visualization with the use of virtual 3D models or specialized geographical information systems (archaeology-oriented GIS). Archaeological objects of the Stone Age are "merged" with the objects studied by geologists, paleontologists, palinologists and other researchers, which in itself creates a unique combination of the paleoecological and archaeological features. Modern research methods require a high level of accuracy and reliability in the 3D fixation of the cultural layer elements, particularly in a comprehensive research of the Stone Age sites. Time gradually destroys the traces of human activity and it has to be admitted that only a minimal number of the Stone Age sites can be actually studied.

When talking about the time frame of thousands of years it has to be remembered that wood, skin and other materials (except for stone and bone), which could have really served as raw materials for the production of work tools and art pieces, didn't remain intact. Climate and terrain have changed many times during the thousands of years, and this has definitely influenced the nature of deposits containing artifacts and traces of other paleocultures. Therefore, excavation technology, used in the case of Stone Age objects, presupposes the use of the work package. There is a need for an exact 3D allocation of the various research objects (research objects referring to various areas of knowledge). The presence of a small portion of the Stone Age human activity remains urges the researchers to use the thorough recording methods, when dealing with the excavation data. It is impossible to carry out proper up-to-date excavation marking and material fixation without adequate geodetic support. Highly effective equipment and specialized methods are required in this type of work.

The terms "thorough recording" and "up-to-date accuracy requirements for recording" seem be of abstract nature in the given context. Metrological support is designed for adequate ranging of any necessary land measuring procedures conducted on the archaeological objects. In other words, metrological support stands for "the application and use of scientific-technical and organizational foundations, technical means, rules and standards in land measuring, surveying and cartographic work" [OST 68-14-99, 2000]. The issue of "required accuracy" remains open. These issues can be solved only after a special analysis of metric and semantic data has been carried out for each type of archaeological observations. Therefore, the required analysis, organization, execution, and control over archaeological geodetic projects have to be under the jurisdiction of the special division in geodesy – archaeological geodesy. Archeological geodesy is responsible for the interrelationship between scientific, technical, and industrial work, which is concentrated on positioning or survey of the archeological objects and on tracing them in the historical time scale.

The assignment of archaeological geodetic work assumes the process of metric and semantic recording for the archaeological objects. Archaeological research is directed at the study of ancient objects. These objects are of particular nature and therefore archaeological geodesy has to deal with the specific information resources. The time factor is reflected in the information resources as the cause of the recorded position of the archaeological and natural objects, and also as the cause of their 3D relocation during the historical time span. Some land measuring and land surveying methods have been used by archaeologists for quite a while now, but there were no real scientifically-based instructions and standards. This publication was supposed to highlight most vital research objectives:

- the conceptual framework and archaeological-geodetic work package had to be reviewed;
- principles of accuracy in archaeological geodesy had to be studied;
- there was proposed an effective method, where various technical elements of applied geodesy were well-combined, for example positioning of the archaeological objects in the required frame of reference, pegging out of the physical 3D survey network, and detailed executive survey in the process of excavations.

2. GOALS, TASKS, MATERIALS AND METHODS OF THE RESEARCH

The goal of the research was to develop the optimal set of geodetic operations for the field archaeological research in Gorny Altai.

The research tasks were:
- to study the conceptual framework for archaeological geodesy, the particular features of geodetic support in archaeological research at the Ust-Kanskaya Cave site;
- to develop the efficient technique of geodetic support for the 3D physical survey network at the archaeological excavation site;
- - to develop an efficient technique of geodetic fixation of the general location of the archaeological site.

The publication was based on the results of the special land measuring, which was completed in the period of time from 1996 to 2003 on the multi-layer sites of Gorny Altai: Karama, Ust-Kanskaya Cave, Denisova Cave, Ust-Karakol-1, Charyshski Cave and several other sites.

These sites have been studied for many years by the researchers from the Institute of Archaeology and Ethnography of Siberian Brunch of the Russian Academy of Sciences under the leadership of Professor A.P.Derevianko. Suggested research methods were tested on the special research testing area near the Ust-Kanskaya Cave [Postnov, Vergounov, 2002].

During the testing, various geodetic instruments were used. Various technical elements of applied geodesy were used as problem-solving methods: fixation of the archaeological site's general location, pegging out of the 3D physical survey network at the excavation sites and conducting the detailed execution survey of the excavation process.

3. RESULTS OF THE RESEARCH

3.1 The Conceptual Framework and the Archaeological-Geodetic Work Package

In the course of the research it became clear, that in terms of industrial process, archaeological geodesy represents a work package, which includes: land surveying, land measuring and specialized geodesic projects, which were carried out during the research of the archaeological sites. Although various geodetic applications are commonly used in archaeology, the present publication deals strictly with the land-measuring project at the Ust-Kanskaya Cave site.

3.1.1 Archaeological-Geodetic Work

In accordance with the research results it is possible to define the following operations in archaeological geodesy:

- land measuring (applied archaeological geodesy) results can be found in catalogues of coordinates and elevation numbers for the archaeological objects and the astronomically important azimuths of archaeological objects;
- land surveying (archaeological topography), which is based on the archaeological survey results in the originals of the situational plans, which contain the archaeological specifics of the situation and relief (archaeological plans);
- topographic-geodetic production, based on land measuring and land surveying results in the course of archaeological objects research;
- archaeological GIS production is based on development of the analytical process and presentation of 3D archaeological localization data. GIS production results in the information about certain archaeological objects, their combinations, and additional information resources for global and regional modeling of the paleoecological processes.

3.1.2 Geodetic Work

The research results show that types of land measuring in archeological geodesy include series of operations in applied geodesy, primary land measuring, and specialized land measuring:

- creation of the basic survey network in the area of archaeological field research;

- pegging out of 3D physical survey network of excavation in-situ;
- projection in-situ of system axes and elevation numbers on the excavation horizons;
- executive detailed survey of the excavation process;
- positioning of the archaeological object in the frame of reference of another object or in the State frame of reference;
- reconnoitering and inspection of the geodetic or basic network in the area of archaeological field research;
- positioning of the points by using satellite, land measuring and astronomical methods;
- preliminary data processing of the satellite, astronomical and land measurements and strict measurement data post-processing;
- geodetic support of the field archaeological research and evaluation of the generalized corrections, caused by the regional deformations of the State geodetic network.

3.1.3 Summary

As we can see, there is a wide range of projects in archaeological geodesy and it isn't limited only to applied geodesy or specialized land measuring. Furthermore, due the fact that nowadays archaeologists have a fare chance of using the data, obtained through the application of high-tech geodetic methods (remote sensing, laser scanning, spectrozonal satellite survey, satellite positioning). These materials also play the important role in the complex study of the archaeological sites.

3.2 Specifics of the Research Conditions

3.2.1 Structural specifics

The analysis of traditional schemes of accuracy enhancement in the course of geodetic work has shown the inadequacy of its direct application in the archaeological field research. The accuracy control in archaeological geodesy is opposite to the one adopted in the construction of the State Geodesic Network (SGN). The principle of State Geodesic Network (SGN) is in the application of a more precise geodesic network in the projects of less precise land measuring. The principle of precision in archaeological geodesy means the application of less precise geodesic network in the projects of more precise archaeological land measuring.

3.2.2 Applied (archaeological) specifics

The study of materials which have been obtained in the course of archaeological field research shows that archaeologists normally work with 3D site models. In order to carry out this type work, there is a need in close monitoring of the excavation process. This type of monitoring has to be conducted in the form of executive layer-by-layer survey, which is done by using special 3D network of coordinates. Also, coordinates of the artifacts correlated in a special way with the complex research materials, have to provide additional information about chronological indexing of the site.

3.2.3 State specifics

The study of the standard technical materials, which regulate archeological field research shows that there is a lack of the brunch financial support for the archaeological-geodetic projects. The state financing of the scientific-research projects is based on the short-term grants, in which expenses for the archaeological-geodetic support aren't even appropriated.

3.2.4 Local specifics

Ust-Kanskaya Cave is a well-known, multi-layered Russian Stone Age site. The site is situated in the Charysh River valley in the Gorny Altai. This area is difficult to access and is not well-studied from the geodetic, geomorphological, and geological points of view. The road and communications infrastructure has not been developed in this area. There are no local land measuring and land surveying brigades in the Ust-Kansky area. There is no State land cadastre network, and there is no real single regional geodetic network. Certain land measuring and land survey projects were conducted in different local frames of reference. The use of such geodetic data is not expedient for the purposes of archaeological geodesy. The distance from the site to the nearest SGN points (State Geodesic Network) is normally 7-25km along the straight line. Usually, SGN points are erected on the commanding heights with marks 1.4-2.5km, which can be approached only by mountain-climbers and helicopters.

Ust-Kanskaya cave is located in the steeply sloped relief area in the massif of marbled Silurian limestone (1084м above the sea level and 54м above the shore line of the Charysh River). The cliff massif itself is called "White Stone" and represents a tectonic block in the field of Cambrian-Ordovician peach stone, in the area of deep long-lasting Charysh-Terektian deep break with a high tectonic activity – 2-3 earth tremors per year (up to 5 on a Richter scale).

3.2.5 Summary

In the study of the Russian Stone Age sites, archaeological geodesy has its own particular features; therefore, the formal application of execution standards used for other purposes, leads to unfounded input of time, effort, and means. The study of traditional schemes of accuracy enhancement for the geodetic projects has shown the inadequacy of its direct application in the archaeological field research. It is necessary to develop new, scientifically-based principles of accuracy control and organization of land measuring, which are used for the purposes of archaeological research.

3.3 Geodetic Support of the 3D Physical Survey Network of the Excavation

Modern archaeological technology of site research includes various methods, approaches and techniques directed at the study of the cultural layer, with consideration of its stratigraphic position in profile, planigraphy, and level of preservation. Standard requirements for archeological land measuring are specified primarily in accordance with the corresponding accuracy standards. These standards determine the selection of the methods and development of the rational methods of work organization and execution.

The research results have shown that in the case of archaeological geodetic operations at the excavation site, the foundations for precision standards calculation are as follows:

- the parameters of excavation structural units;
- the size of the site and the need in a coordinated orientation of the structural units' totality within the site itself;
- the need of correlation between the location of the studied site and other objects and also the distance between the objects and the site.

3.3.1 Structural excavation unit and 3D physical network of coordinates

The analysis of the excavation process shows, that depending on the research task and type of research, the bedding of the cultural layer, the researcher sets the structural unit of excavation, which is determined in accordance with:

- the minimal size of the excavated site area (sampling area), where the traces of human activity and the nature of their location in the lithological horizons are recorded after the layer was studied;
- sampling thickness (sampling depth) of the excavation sampling area within one lithological horizon: sampling levels are numbered top-down.

The structural units are individually selected for each site. The size of a structural unit is stipulated by the saturation of a cultural layer and by stratigraphic bedding of lithological horizons. In the course of excavations the following thicknesses of the sampling level are accepted: from 2cm up to 30cm.

The following standards have been applied to the Ust-Kanskaya Cave site. The square with a side length of one meter and sampling level of 5cm was chosen as a structural unit. There was a plan for each level, even if the finds were not recorded during the clearing. The amount of excavated artifacts at the site could be as high as 100 samples from one structural unit. For convenience in setting up of the 3D survey network and in order to avoid errors in reading from the z-axis, the height of a "cell" was assumed to be 1m. In office lab such cell height allowed to easily recreate the field reading, with the error caused by omission of the total number of cell heights.

The 3D physical network of coordinates was limited by the cave’s geometry and couldn’t exceed $15*4*3$ meters along the axis of *X, Y and Z* respectively. At the beginning of each of the field seasons, the network of coordinates was restored and expanded. In the year 2003 network of coordinates was $13*3*2$ (in meters).

For land measuring at the excavation site, the relationship between the values of root-mean-square error *(RMS error, μ)* and limit absolute error *(limit error, m)* was $m = 1.96 * \mu \sim 2*\mu$

(reliability level 95%). The tolerance limit error was marked as *"M"*. By using the term "reliable" in the text of the present work, the authors mean 95% reliability level. This means that in order to control the quality of land measurements (in other words for correspondence of their reliability to the selected level) not less, than 20 new measurements have to be completed in the future. At the same time it was acceptable to get deviations d with the controlled meanings:

- no less, than for 85% control measurements - values $d < 2 * \mu$;
- no more, than for 10% control measurements - values $d = 2 * \mu$;
- no more, than for 5% control measurements - values $d > 2 * \mu$.

3.3.2 3D marking (three-dimensional physical network of coordinates)

Rectangular coordinate system was used for marking of the excavation area. Abscissa, ordinate and z-axes meet in the point of origin, which is designated outside of the excavation margins. This was done this way in order for an excavated area to have positive domain values of abscissa and ordinate axes. Ordinate axis was increasing in the northern direction and abscissa axis was increasing in the eastern direction. *Z*-axis was directed up, the heights and depths at the excavation were counted out from the zero mark of the *Z* -axis. The coordinates of the point under survey, which corresponded to the values in meters in a frame of reference, gave a numeric definition to the square *(1m x 1m)*, which was situated in the increasing absolute values along the abscissa and ordinate axes. When the squares were numbered, first the abscissa and then the ordinate values, were specified (e.g., square *3*-*8*). Originally, the line directions were determined with reference to the magnetic meridian. Later, in 2003, the network of coordinates was oriented relative to the true meridian. On the excavation site, the suspension and ground marking systems were set up and the area was divided into squares with 1m-long sides. Two marking systems were coordinated through the measurement of elevations and difference of plane coordinates. The marking coordination control and the coordinate network quality were checked before the actual start of measurement process, (no less than twice a day – before the morning shift and in the evening before the evening shift).

Metal wire, used for suspension marking, was getting deformed due to the effect of tension. It was important to correct the suspension marking. The suspension marking was carried out by using non-intersecting kapron threads and the tension was created by using special accessories, made of tambuckle and wooden bracing. Kapron threads were fastened to the metal wire *0.6mm* in diameter, stretched between the special anchors. Corner anchors, which were *12mm* in diameter, were designed for maximum high-duty applications and were put in the cliff foundation at *120mm* deep (*200mm* in case of loose rock). Anchors with a diameter of *10mm*, which were used to press the wire to the walls of the cave, were put in the cave’s foundation *70mm* deep. The wire was stretched by using winches with an exertion in the order of one ton.

A frame of reference was anchored to a special point (zero reference point), which had three centers in the form of rocky marks with an original structure for enforcement of axle centering. In this case, the centers were the holes, manually drilled in a hard rock (Silurian marbled limestone). These holes were *50mm* deep and 6mm in diameter. Wooden plugs were driven into each one of the holes. There were metal rods in each one of the plugs (rod diameter – 1mm). These rods ended outside with metal loops. Wooden plugs made it possible to easily set out all three of the metal rods, and to adjust their loops in a single horizontal level at the time of coordinate network creation (at the beginning of excavations in 1999).

The horizontal level elevation number specified in 1999 by zero reference point centers, had a "*-1m*" value. Due to the fact that this horizontal plane was shifting along with rocky massif, it recorded the relative frame of reference and excavation elevations in a more reliable way (in case of various deformations and shifts of the cliff's foundation). For the archaeological research, a rigid fixation of the frame of reference was used at the time of excavations. This is normally more important than the accuracy of absolute plane coordinates and elevation number of the zero reference point in the State frame of reference.

By using kapron threads, which were stretched through the loops in the zero reference point centers, laser or optical level, hand laser ranger or ordinary geodetic tape-line, it was easy to recreate the originally specified reference level of the excavation's frame of reference (reference zero-level). One of the center-connecting threads went exactly through the *25m* mark on the ordinate axis. The continuous everyday control was required for the currently marked physical coordinate network and the everyday correction of the suspension marking system was carried out in accordance with the plane specified by these threads. Right on the cliff, near each of the marks, there were markers – white plates *(5x10cm*), where the exact coordinates of the mark centers were specified. All of the three marks were fixed in a single horizontal reference level. The heights and depths of the excavation were counted off from the zero-level using that rigid fixed reference level.

So, the zero reference point was safely fixated by the three cliff marks and any researcher in the future could easily recreate the 3D frame of reference. The center of the State Elevation Network reference point was used as a reserve excavation elevation reference point, located at the bottom of the "White stone" mountain: the place where the cave itself was situated. The State elevation number of the zero reference point of the excavation was related to this State reference point. This would give a chance to recreate the elevation marking, in case all three centers of the zero reference point were lost.

3.3.3. Accuracy design for pegging out of the Z-axis

The metrological criterion of insignificant values was used. The location of the lower mark of the sampling level was tied (for the single elevation number measurement) to the zero-level of the excavation with tolerance limit error no more than $Mz = K * 50mm$, where K is a numerical meaning of this metrological criterion. It is a common knowledge that the principle of insignificant values provides such work organization where the errors of the previous project stage (location of the sampling level borders) should not influence the accuracy of the current project stage (integral analysis of the whole layer). The value $K = 0.3$ served as a metrological criterion, adequate for the selected reliability level. So, the tolerance limit error of the layer border location was $Mz = 0.3 * 50mm = 15mm$. If the summary errors of the level border location weren't exceeding *15mm*, their influence on the procedure of the level analysis constituted a value of $(12 + 0.32)1/2 – 1 = 0.044 \sim 4.5\%$ from the thickness of the sampling level. This error level was ignored for those purposes.

In the course of excavations, it became clear that fluctuation of the sampling level border in the range of *1cm* with its thickness of *5cm* was a relatively common phenomenon ($m_{of\ the\ level} = 10mm$). The reading with the use of a physical 3D coordinate network was conducted by two observers in orthogonal planes, and the measuring rod was also used in that process. This reading made it possible to work with RMS error, the value of which was not higher than the double diameter of the thread of the physical coordinate network. At the Ust-Kanskaya Cave, a kapron thread with diameter of *0.5mm* was used for the coordinate network. From this, it follows that limit error readings of height/depth (in relation to the upper plane of the slave cube of the coordinate network) was $m_{reading} = 2 * 2 * 0.5mm = 2mm$.

Z-axis for the 3D coordinate network was pegged out with the use of laser level (direction indicator) "Limka-horizont" with the ray reversed into a plane by using cylindrical prism (addition to the object-glass "Limka-Line"). RMS error of elevation measuring was *3mm* per *10m* and limit error vertical pegged out of the coordinate network by using the method "from the middle" correspondingly didn't exceed the value of $m_{pegging\ out} = 2 * 0.3мм * (1.5^2 + 7.5^2)^{1/2} \sim 5мм$. From this the limit error was calculated:
$m_z = (m_{pegging\ out}^2 + m_{level}^2 + m_{reading}^2)^{1/2} = (5^2 + 10^2 + 2^2)^{1/2}мм \sim 11мм$, which was significantly lower than the value of the tolerance limit error: $m_z = 11mm < M_z = 15mm$.

3.3.4. Accuracy design for pegging out of abscissa and ordinate axes

The issue of measurement accuracy of the artifact plane coordinates in the excavation of the frame of reference is very important in the archaeological research, but it is of secondary importance in relation to the question of the z-axis measurement. Therefore, it was assumed that the measurement errors in relation to the reference point on all of the three axes were of the same degree, $M_x = M_y = M_z = 15мм$. Then the value of the tolerance limit error of the artifact location in the frame of reference of the excavation was $M_s = (M_x^2 + M_y^2)^{1/2} = 1.41 * 15мм \sim 21мм$ (M_s – tolerance limit plane position error). The value of limit plane position error m_s was made up of limit error of the linear pegging out of the abscissa axis m_x and ordinate axis m_y, limit error of pegging out of right angle between the coordinate axes m_β and limit error readings on both of the system axes: $m_s = (m_x^2 + m_y^2 + m_\beta^2 + m_{x(reading)}^2 + m_{y(reading)}^2)^{1/2}$.

The reading error was already determined during the review of accuracy level in the process of the z-axis pegging and its obtained value was used in our case for abscissa and ordinate axes $m_{x(reading)} = m_{y(reading)} = m_{reading} = 2mm$. The linear pegging out was conducted by using the laser tape-measure, RMS error of the *100m*-long distance measurement constituted a value of $\mu_d = 1.5mm$. Due to the fact that on the shorter distances the effect of the systematic error (the range-finder constant) on the measurements was more significant, than the influence of all random errors, then it was possible to consider the linear limit error of pegging out of the lines with length of up to *50m* to be a constant value, $m_x = m_y = 2 * 1.5mm = 3mm$.

The limit error in pegging out of right angle between the coordinates of the axes depended on the type of instruments that used. The quicker, but less accurate pegging out of right angle from the middle of the long side was done with the use of the optical inclinometer (Suunto Tandem model, Finland, which consists of inclinometer and geodetic compass). After that the obtained location of the point was adjusted by ranging the diagonals, which reached the ends of the long side. The slower version of the pegging out process was based only on the measurements conducted by using laser range-finder and the availability of the high number of network squares for the direct measurement of their sides and diagonals. RMS error of the reading by using an optical microscope on the scale of the horizontal circle of inclinometer constituted a value of $\mu_\beta = 0.1° = 6'$. Therefore, the limit error value of the pegging out of right angle between the coordinate axes in the linear units was $m_\beta = 4000mm * 2 * \mu_\beta° * \pi / 180 = 14mm$.

When doing the pegging out of the right angle by using only range-finder for the multiple cross ranging, where the positioning point was determined by the three "stations": beginning from the long side, end of the long side and its middle. Then limit error didn't exceed the value of $m_\beta = 2 * \mu_d * (3)^{1/2} / (2)^{1/2} / \sin 62° = (1.5)^{1/2} * 3mm / \sin 62° = 4.2mm$. In our case, the cross-ranging angles were congruent and equal to $\gamma \sim 62° (\cos\gamma = (8.5^2 + 4^2 - 7.5^2) / 2 / 8.5 / 4)$. When there was conducted pegging out of the right angle from the middle of the shorter side, the cross-ranging angles were equal $\gamma = 7.6° (\cos\gamma = (15.1^2 + 15^2 - 2^2) / 2 / 15.1 / 15)$, and the value m_β was calculated $m_\beta = 2 * \mu_d * (3)^{1/2} / (2)^{1/2} / \sin 7.6° = (1.5)^{1/2} * 3mm / \sin 7.6° = 27.8mm$.

It is quite possible to encounter such a geometry of the excavation where in order to do the pegging out of the right angle, it was not enough to have an optical inclinometer which operated with precision (the cross-ranging lines were too long) and the linear measurements which had to be carried out for the ends of the long side were complicated by the man-made and natural obstacles. Then the theodolite or tacheometer had to be used.

The $m_\beta = 14mm$ value was taken as a limit error value for pegging out of the right angle and the limit error value was calculated and compared with the legitimate value of $M_s = 21mm$. So, $m_s = (2 * 3^2 + 14^2 + 2 * 2^2)^{1/2}\ mm = 15mm < 21mm$. In this case the traditional equipment wasn't required (cumbersome optical theodolite or expensive electronic tachometer).

3.3.5 Summary

Excavation technique is a decisive part of field research in archaeology. In the course of excavations there should be an attempt made to obtain various types of information from the sources, in order to comprehend the wide range of information. This principle of the complex analysis covers not just the artifacts or the material culture, but also the environment of this material culture. In the excavation process the one has to think not about the removal of the cultural layer, but about its detailed study. Flinders Petri was one of the first researchers, who formulated the principle for one of the main methods of archaeological fixation of artifacts:

- to preserve as much as possible at the site;
- to collect all of the material which can't be preserved at the site;
- to fixate "in situ" everything more or less significant: descriptions, measurements, technical drawings, sketches and photographs;
- to create the complete report about the excavated site, in order to make all the facts available to the public as soon as possible.

By applying their lengthy field work experience, venerable archaeologists can easily design the set of accuracy parameters, necessary for excavation, in accordance with the nature of each of the sites. This type of design process is much more difficult for the young specialists, for whom various sets of accuracy parameters turn to be a dogma. The application of such "standard" set of accuracy parameters in completely different conditions doesn't always allow to attain the preset goals. The example given above illustrates that based on the suggested methodology it is possible to carry out all required precision calculations for each of the sites. These calculations will be quite sufficient in order to carry out the geodetic projects at the excavation sites. The cultural deposits were approached along the individual squares of *1m x 1m* and *5cm* deep within the same lithological layer. The artifacts detected during the study of the cultural deposits were divided into two groups. The first group included:

- stone tools
- primary stone splitting products with a size larger than *2cm*
- definable fragments of large mammal bones
- indefinable fragments of large mammal bones with a size over *5cm*

The second group of the artifacts included:
- primary stone splitting products with a size less, than *2cm*
- indefinable fragments of large mammal bones with a size under *5cm*
- rodent bones

In the process of the square and level working of the cultural deposits, the finds of the first group were left intact at the excavated surface of the layer, and the finds of the second group were collected and marked after the fixation of their coordinates. Detailed metric and semantic characteristics of the excavated level were recorded in the field documents. After that, the finds of the first group were entered onto the layers plan as notation conventions. In order to process the data by using the computer, the table was put together, where each of the finds was entered in the horizontal cells and their features were entered into the vertical columns. The following columns were prepared for the artefacts of the first group: the site, year of excavation, layer, square, number, name, 3D coordinates, the position (side facing upwards), horizontal and vertical orientation. In the course of the office studies, the set of features was added by petrographic, functional and morphological characteristics. The electronic version of this table constitutes the basis for the GIS-system for the Ust-Kanskaya Cave and Charyshsky sites.

Individual code was assigned to all of the recorded finds except for bone fragments, stone artifact fragments, debris, split pebbles, found in the course of the dry screening and ablution. The number (except for the unclassified objects, found in the course of dry screening and ablution) was assigned to the finds based on the layer-to-layer system of through numeration within a single square. Each of the finds had its individual code which was based on the site's name, number of the find, number of the square and number of the layer and the code also depends on the location: excavation or screening/ablution (the finds with screening/ablution have a letter *"P"* in the index). The horizontal and vertical orientation for the finds was conducted with the consideration of the plane length no more than 3cm. The plane hade was measured for the plane, where the find was located relative to the horizontal plane of the excavation frame of reference with precision of up to *5º*. Horizontal orientation was determined from the ordinate axis counterclockwise to the direction of maximal inclination of the plane, where the find was located (in accordance with a horizontal angular scale of mining compass). The vertical orientation of the find was determined in accordance with the vertical goniometrical scale of the mining compass as a maximal angle between the horizontal plane and the plane, where find was located. The location depth of the finds was indicated in centimeters relative to the conventional zero level of the excavation.

All of the excavated soil was sent for screening and ablution. The molehill filler was taken out screened and/or was put through the ablution process. The archaeological and faunal material wasn't fixated on the general plan of the layer, however it was considered in the general characteristics of the finds on the layer-to-layer basis together with materials of the first and second groups.

3.4 General Sites Positioning

3.4.1 Getting the elevation marks (excavation elevation numbers)

The absolute elevation mark of the State Elevation Network reference point center, which was put in the foundation of the cliff massif (where the cave is located), was *1,030.823m* (Baltic elevation system of 1977). The elevation value between the *"-2m"* level in the excavation frame of reference and the reference point (which results from the trigonometric leveling by using electronic tacheometer Pentax II), was *+51.534m*. The obtained elevation data are given in Table 1. It was considered that:

- there was a strong tectonic activity at the site (which leads to changes in elevation of the massif during the lengthy time period),
- there was an error in projecting of the elevation onto the excavation,
- there was an error in 3D coordinate network setup.

So, it could be assumed that the absolute elevation level of the *"0m"* of the excavation was determined with precision of up to *1cm*. The accuracy requirements in determination of the absolute elevations are determined by the terrain microform sizes, which reflect the geomorphologic features of the river valley and which have an important meaning for the archaeological research. Therefore, the information about the excavation zero-level elevation mark in the State Elevation System presupposed the use of the reference point (with centimeter accuracy) in the State Elevation Network. The reference point served as a reserve center for recreation of the excavation elevation system (with loosing of all three centers of the zero reference point).

Table 1: Elevation data from the Ust-Kanskaya Cave

Station	Elevation numbers in system BES-77	Elevation numbers in system of the excavation
Reference point	1030.823m	-53.534m
Elevation	+51.534m	+51.534m
Level "*-2m*"	1082.357m	-2.000m
Elevation	+2.000m	+2.000m
Level "*0m*"	1084.357m	0.000m

3.4.2 Project and the program of navigational satellite positioning

- The forecast of the satellite observation sessions produced the schedule of the working sessions (in accordance with which in summer of 2003 the most favorable were the three intervals 60-90 min each in the three sessions – №4, №6 and №7). These sessions were observed (in the following intervals): 13^{15}-14^{59}, 17^{15}-18^{59}, 19^{15}-20^{59} (local time).

- The trial satellite survey was carried out in various ways in the sessions, characteristic in terms of their quality (mountain peaks) for the determination of the most appropriate methods in satellite positioning. The absolute positioning (with consideration of the local corrections to the orientation of the earth ellipsoid) and the relative positioning between the two separate points were of the same quality *(RMS error=3.4m)*. However, they were characterized by various accuracy degrees (around 4 and 2m accordingly). The most suitable method for this area was selected – relative positioning at the points of a single net of the determinate points.

- With the use of the two navigational GPS-receivers Garmin Rino 110, the absolute values of the plane coordinates and of the base survey stations were obtained based on the relative satellite positioning (the increment of coordinates for the observed points in relation to the base survey stations). Simultaneity of fixation of the position of both observers was supported by the radio communication between them within the VHF range. The systems of obtaining differential corrections for the navigational receivers (with the use of European or US geostationary satellites) weren't available in Gorny Altai in the summer of 2003.

- The measurement program assumed the increment of coordinates up to each one of the points from the determination of one base line and several connecting lines (the lines connecting the point with the base line and other secondary points). In case there were no connecting lines the determinate point was treated as "uncontrolled" (cantilever extension). When there was only one connecting line, the coordinates of the determinate point happened to be the average-weight values, obtained by using base and connecting lines. Measurement of the two or more connecting lines gave a chance not only to evaluate the accuracy of the measurement itself but also to exclude the review of the line with major errors. The repeated satellite survey of high accuracy (*RMS error* $< 1.5m$) was at the State Geodetic Network points, and the base survey stations and base points gave the parameters of connection with the State Coordinate System.

- Observations at the determinate points: in places with more favorable location for the satellite observations, a high level of accuracy was achieved, *RMS error* $<3m$; in places with less favorable location for the satellite observations, *RMS error* $<5m$. In the process of satellite measurement filtrations, out of 342 definite lines (including trial and repeated observations, reoccupation) there were 110 lines left for the further analytical treatment (taken for the final equalization). The analysis of the systematic errors in line measuring (consistent discrepancies in the readings of the two navigational GPS-receivers, based on the measurements in the area) gave the values of $Cx = 5cm$ *и* $Cy = 7cm$, which allowed to disregard these values during the calculations at all of the areas.

- In order to conduct an independent control of the satellite measurement network equalization, the planned, ground-based, instrumental line measurements were conducted between the conveniently located base survey stations. All measurements were reduced onto the surface of the cartographical six-degree right angle projection of the Gauss-Krueger (Transverse Mercator projection). The comparison of the measurements of the controlled lines (between the base points) were made by using laser tape-measure with their values (calculated in accordance with the equalized values of the base survey stations' coordinates), gave the maximal deviation of less than *50cm*. The accuracy of the satellite positioning was later indicated by strict post-processing of the ground-based trilatteration data. Station coordinates obtained in the course of the satellite positioning were used in post-processing as the approximate position of the points in the ground-based network. Probable corrections in the station positions obtained in the course of strict trilatteration post-processing were in the range of *24-61cm* and only in one case the correction constituted 85cm.

- The accumulation of data, their visualization and analysis, and equalization calculations were conducted by using engineering software for the calculator "Cassio Algebra 2+". The software has been specifically developed by E.Vergunov for the post-treatment of the navigational satellite observations during the field work (in the real time). The program was automatically selecting data from the electronic observation diary: it produced the list of the base survey stations, the base and connecting lines (and their weights), and prepared the normal equations, solved them, and produced an accuracy analysis and the search for the systematic errors. The satellite network was equalized as a "fluctuating" network: in other words, location correction has been applied to every station and/or point. The iterative equalization process was applied, where filtration of the major errors in the satellite measurements was happening in accordance with the method of statistical funnel [Postnov, Vergounov, 2003].

– Statistical funnel: In the systemic-dynamics there exists a notion of a funnel: the class of solutions in the case of which the decision space is getting smaller and smaller. Such an approach in the geodetic application gives a chance to "filter" the executed satellite data (results) in accordance with their accuracy (by using the statistical funnel). In case "at the bottom" of such a funnel there remains incommensurably few data of the required accuracy in comparison with the input effort, then the used field work methods (the conditions and structure of the observations, used equipment, qualification of the worker) aren't appropriate for finding a solution for the problem. In our case, the filtration of the defect was carried out by re-equalization of the whole satellite network, and by adding of every new determinate location or point to the ones that have already been processed. The measured lines with equalization corrections of which exceeded a certain degree of tolerance were specially marked. The degree of tolerance is determined by the number of equalized measured lines. Only one of the marked lines had to be immediately excluded: the one with the highest correction as a result of equalization (in accordance with the absolute value). The next equalization was carried out without that excluded line. The degree of tolerance was changed in accordance with the new number of the equalized lines. The correction from the new equalization was comparable with the new tolerance degree value. This cycle of line equalizations and exclusions was repeated until there were lines, the corrections of which exceeded the adequate degree of tolerance. The statistical funnel is characterized by its specificity. At the same time, it is characterized by a high sensitivity level. The statistical funnel doesn't allow the measurements with major errors and only the measurements with high error level have chances of being left out. At the end of the filtration process, if the "leave out list" is empty, all the remaining repeated line measurements are excluded – each line has to have only one measurement. Due to the fact that the question of determination of the accurate value (for one or another satellite navigation survey) is in the stage of development, it is not recommended, at the present time, to select the average-weight value out of several satellite measurements (for the determinate line out of several satellite surveys).

3.4.3 The project of the ground-based instrumental coordinate measurements

In order to have the azimuth for the axes of the excavation frame of reference there was carried out a y-axis projection into the Charysh River valley and its fixation with the temporary reference points (TRP). Highly accurate measurement of the slant range and of the slope angle along the line from the temporary reference points to the center of the excavation zero reference point allowed to obtain the coordinates in the State Frame of Reference for all three centers of the zero reference point. Unfortunately, it was impossible to anchor the azimuths of the excavation coordinate axes only by using the zero reference point centers because of the small distances between them. It was also difficult to anchor the direction azimuth from one of the zero reference point centers to the oriented location in the Charysh River valley because of the cave's geometry. In addition to that, there has to be an agreement with the local authorities to lay out the foundation for the basic center (with depth no lower than the ground frost penetration level) in the area of the very valuable agricultural grounds for the mountainous region. Therefore, anchoring to the State Frame of Reference was carried out by using base and orientation stations located outside the cave. Elevation reference point of the State Frame of Reference was chosen as the base survey station (BSS). Its center was anchored by the cliff mark at the bottom of the "White Stone Mountain" and could serve as a compilation reference point for the excavation frame of reference. The security concrete pole located on top of the underground telephone line was chosen as orientation station (OS) for the direction (BSS-OS) azimuth fixation. Ranging line, along which the ordinate axis of the frame of reference was projected into the Charysh River valley was laid out no rougher, than $m_D = 10mm$ with its full length of $D_D \sim 600m = 600000mm$. The errors of the most probable location for the basic survey network locations were $m_{s\ TRP\text{-}10} = 1.6mm$ *и* $m_{s\ TRP\text{-}19} = 4.5mm$, $m_{s\ BSS} = 3.1$*мм и* $m_{s\ OS\text{-}12} = 1.1mm$. The values $D_{[TRP\text{-}19]\text{-}[TRP\text{-}10]} \sim 300m = 300000mm$ *и* $D_{[BSS\text{-}[OS\text{-}12]} \sim 270m = 270000mm$ were taken for calculations as the approximate values for the line lengths between the locations. In that case, the azimuth attachment of the excavation frame of reference to the State Frame of Reference was carried out with the final error of

$m_{\alpha DL} = (m_\alpha^2 + \rho''^2 * m_D^2 / D_D^2 + m_L^2)^{1/2} \sim (3.4^2 + 4.0^2 + 2.9^2)^{1/2} = 6.0''$, where

$m_\alpha = \rho'' * (m_{s\ TRP\text{-}10}^2 + m_{s\ TRP\text{-}19}^2)^{1/2} / D_{[TRP\text{-}19]\text{-}[TRP\text{-}10]} \sim 206265 * (1.6^2 + 4.5^2) / 300000 = 4.0''$

(the error in the determination of the azimuth),

$m_D = \rho'' * m_D / D_D \sim 206265 * 10 / 600000 = 3.4''$(error in the pegging out of the ranging line),

$m_L = \rho'' * (m_{s\ BSS}^2 + m_{s\ OS\text{-}12}^2)^{1/2} / D_{[BSS]\text{-}[OS\text{-}12]} \sim 206265 * (3.1^2 + 1.1^2) / 270000 = 2.9''$(error in the azimuth attachment),

$\rho^\circ = 180^\circ / \pi \sim 57.3$, $\rho' = 60 * \rho^\circ = 60 * 180^\circ / \pi \sim 3438$,

$\rho'' = 60 * \rho' = 3600 * 180^\circ / \pi \sim 206265$.

However, considering the lack of gravimetric data about the plumming line deviation in the work area (particularly important for the mountainous terrain) and also the fact that the most probable of the basic survey network was obtained from the satellite positioning (accuracy level of up to a meter), the real final error should constitute the value of approximately 20-30″.

In order to evaluate the accuracy of the project, the single scale preservation principle was used, where the tolerance limit error for coordination of the artifacts in the excavation was $M_s = 21mm$. The single scale preservation principle gives us the work organization, where the single value of the absolute error is maintained at all stages and is determined in accordance with the standard requirements for the final result of the accuracy level. The values of relative error are calculated for each of the stages, which will correspond to the assigned absolute error and lengths of the measuring lines. Therefore, in this particular case, for the creation of the basic survey network, it was assumed that $M_s \sim 21mm$. At this point, the traditional approach has to be considered, in which the land measuring standards are formally used for the archaeological-geodetic purposes. The calculation of the operation standard accuracy is conducted in accordance with the required scale. Basic survey network standard requirements can be obtained from the topography, due to the fact that the final result of the physical frame of reference was the fixation of the artifacts on the layer-by-layer plan of excavation. The scale of the plan was 1:20, the limit error of the points' locations for mountainous, forest and difficult in terms of access regions was 0.3mm in the scale of a plan (*0.2mm* for a regular terrain). This gave us a tolerance limit error for the basic survey network points $M_s = 0.3mm * 20 = 6mm$, and the value of the average tolerance limit error for the points of the network was no more, than $M_{s\ average} = M_s / 2 = 6mm / 2 = 3mm$. The land measuring project at the Ust-Kanskaya cave was of a research nature, therefore it has been decided to create the trial basic network with the stations' relative position error equal to $M_s = 6mm$ *and* $M_{s\ average} = 3mm$ by using the portable set of the land measuring instruments.

3.4.4. <u>The program of the ground-based instrumental coordinate measurements</u>

a) With the dimensions of the basic survey network equal to 500m * 400m (Fig. 1), the project presupposed land measuring with limit relative error of 1/83300 (*500m / 6mm = 500000 / 6 ~ 83300*). To achieve such an accuracy level, it was necessary to lower the error level in centering on the network points, provide favorable conditions for the measurement and a good geometry of the network itself. The place for the basic survey network was chosen in Charysh River valley: flood-lands and terrace. Elevation differential between the flood-land of the river and the road-bed was equal to 7m (on the stretch of 250m long), between the road-bed and TRP-19 – approximately 9m (on the stretch of 300m long). The position of the points on the river terrace has provided good conditions for the satellite survey (mountains on the sides of the valley were covering the horizon up to 15º) and also for the ranging measurements (most of the slope angles for the lines were less, than 1º, maximal were 3.5º). The availability of the road Ust-Kan–Ust-Koksa has allowed the use of the road infrastructure elements for the geodetic mini-towers, which were convenient for the land measuring projects with the height above the ground surface from 50cm up to 160cm and the centers of the base survey stations were marked on their upper surface.

In order to create the basic survey network, the method of laser ranging was adopted – micro-trilateration with direct measurement of the distances. Trilateration as a geodetic creation method normally doesn't give many excessive measurements. Therefore, the amount and positional relationship of the basic survey network points in this case had to provide the maximal number of the laser ranging distances, given the good geometry of the cross ranging. The main instrument that was used in measurements was the laser ranging apparatus (without reflection) of the Disto classic make (5th generation). This apparatus enabled the researchers to measure the lines of 700m also by using special targets. To minimize the overall error of the basic survey network, the lines with the length value higher, than 220m were measured by using electronic tacheometer "Pentax II". Land measuring was grouped into three series: (TRP-19 in the agricultural zone) – (T-13, OS-12, TRP-10, T-14, T-15, T-8 и T-9 in the road's detachment zone and zone of the underground cable) – (T-16, T-7, T-17 in the flood-land of the river and BSS – the reference point of the State elevation network). Network points OS-11 and T-18 were excluded due to the lack of visibility (construction of the recreation "Oibok" facilities) for OS-11 and failed geometry for T-18. Therefore, the number of points and locations of the basic survey network: 12, amount of measurements made: 36 (measured sides of the basic survey network), number of the required measurements: 24 (12points * 2coordinates), the number of excessive measurements: 12 (36measurements - 24unknowns).

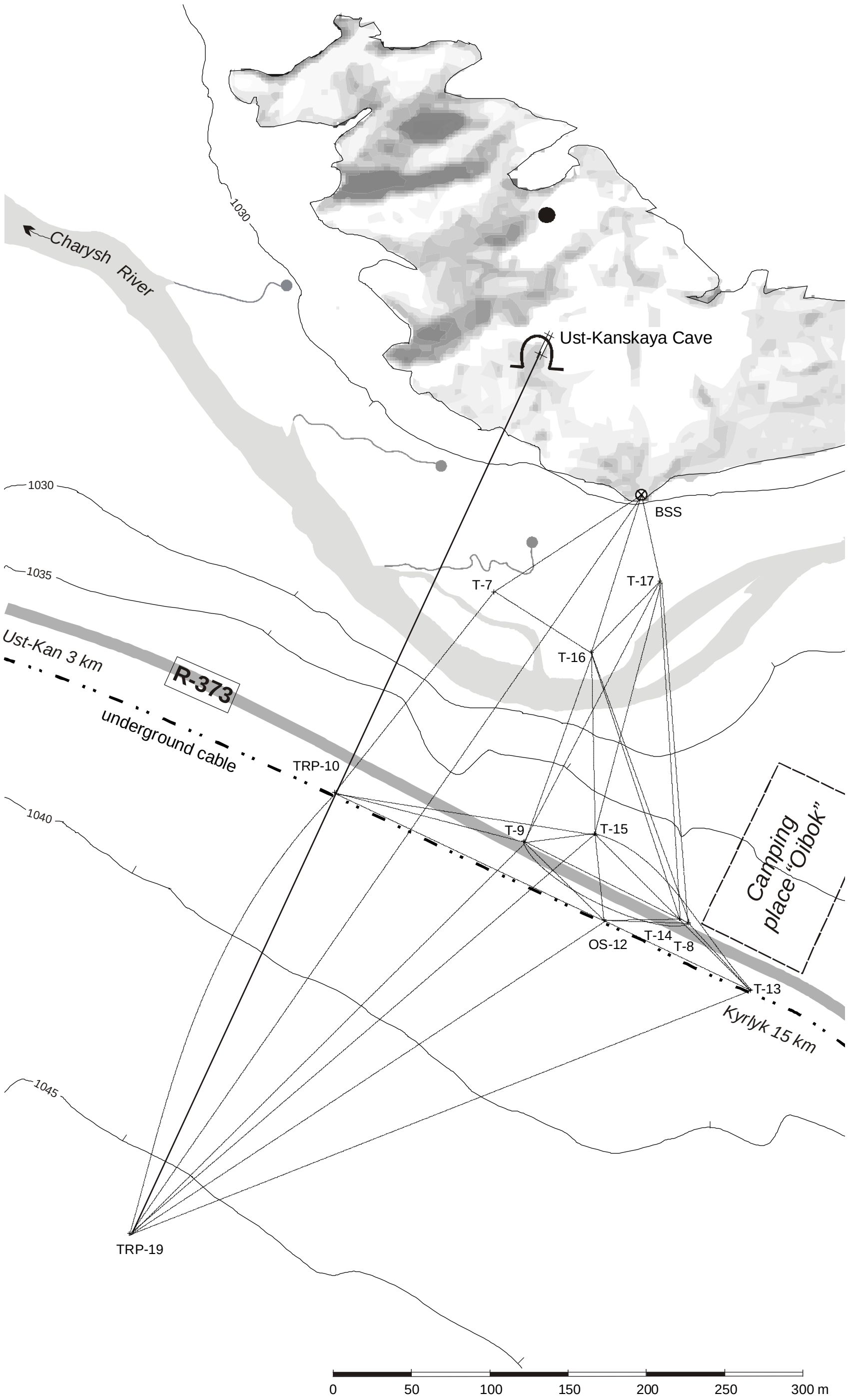

Fig.1: The survey network of the Ust-Kanskaya Cave

The basic survey network was adjusted as a "fluctuating network", i.e. the corrections to the coordinate values of their approximate locations were assigned to all of the network points. The adjusted position values from the post-processing satellite data (with consideration of satellite observations at the SGN point "Southern basis") were taken as the approximated coordinate values for the network points. The obtained weight error is $\mu = 1.1mm$ (measuring error of the side which has a weight of $P = 1$, in our network this is a side of the network, which is 100-220m), average error of the points' relative position in the basic survey network $m_{s\ average} = 2.4mm$, and all errors of the location of the network points, obtained based on the matrix of the reversed weights and weight errors were given in Table 2. Double excess in the error of the relative position of the basic survey network points over the weight error was stipulated by the weak point of the network – TRP-19 (4.5mm). Comparatively high degree of errors in points: t-7, t-17, BSS-8731 and t-16 (2.8-3.2mm) could be explained by high error of centering on them and by 7-meter drop between the terrace and flood-land of the river. The addition of the new points in the calculated locations of the basic survey network would have improved the geometry of the marks as, for example, the addition of the t-17 with the six newly measured lines would have decreased the error in BSS location by 1.5 times. However, due to the fact, that the required accuracy level was achieved ($4.5mm < 6mm$ *and* $2.4mm < 3mm$), then there was no reason to further increase the labor intensiveness of the work.

b) Due to the fact that the instrumental error of line measuring and the methods used for that purpose, allowed to achieve the required accuracy in the plane location of the basic survey network points, the average error of its points' relative position would be determined by the network geometry. The network geometry can be corrected by applying more accurate ranging measurements (particularly in the areas with high error level). Theses corrections are also made by shifting from the points with high error level into a more "correct position" relative to the other points. If both of the above methods aren't feasible, then the new point with its measurements is supposed to be added. Point shifting in this case was not practical, the location of the points should coincide with one of the most convenient local objects, because the appearance of each new point caused real curiosity of the local population or was determined by more solid reasons (BSS and both stations TRP). For the auxiliary points of the network it was easier to exclude this station (OS-11, which was left for the next year's network), or to take another point instead (instead of t-18 to take t-7, t-8 and t-9). The preliminary measurement program presupposed the double measurements of the network sides. The error evaluation for the key station locations (TRP-19, TRP-10, OS-12, BSS) was conducted in order to clarify the measurement preliminary program and there was made a prognosis for the errors of the network points' relative position. The stations TRP-19 and TRP-10 temporarily fixated the projection of the ordinate axis into the valley. Stations OS-12 and BSS fundamentally fixated the azimuth attachment of the excavation's frame of reference.

c) Preliminary calculation of the error evaluation for the station TRP-19. It was enough to calculate single cross ranging accuracy in accordance with a standard formula: $m_s^2 = \sigma^2 (m_1^2$

+ m_2^2**)**, where m_s was the error in the determinate station's location, m_1 – the measurement error of the first crossed line ranging, m_2 – the measurement error of the second crossed line ranging and σ was a coefficient of the cross's geometry: $\sigma = 1 / \sin\gamma$ (γ – horizontal angle at the determinate point between the ranges of the cross ranging). Due to the fact that σ had minimal values at the angles, close to 90º and went up sharply at the angles, which were less, than 30º, the best way of crossing gave us the triangular with peaks at TRP-19, TRP-10 and t-13, and network sides TRP-19-TRP-10 (N22) and TRP-19-t-13 (N25) served as crossed ranges. The m_{22} and m_{25} values were given in the Table 12, the angle $\gamma = 43.7^o$ were measured by using inclinometer.
So, $m_{s\ TRP\text{-}19}^2 = (1.8^2 + 1.8^2) / \sin^2 43.7 = 13.6$ and $m_{s\ TRP\text{-}19} = 3.7mm < 6mm$ (Table.3).

Table 2: Register of the adjusted coordinates for the basic survey network points and errors of their relative position

№	Station or point of the basic survey network, location	X, m	Y, m	m_s
1	T-7, a stake in the flood-land, support point, year 2002. "Island"	518.674	238.930	2.9mm
2	T-8, wooden stake on a wayside	307.530	362.948	1.6mm
3	T-9, wooden stake on a wayside	359.709	257.928	0.8mm
4	ВРП-10, metal pipe in the detachment zone of the underground cable	390.900	137.200	1.6mm
5	ОРП-12, concrete pole in the detachment zone of the underground cable	309.576	309.307	1.1mm
6	T-13, metal landmark in the detachment zone of the underground cable	264.613	402.898	2.3mm
7	T-14, wooden pole on the side of the highway	310.176	357.391	1.6mm
8	T-15, wooden road sign on the side of the highway	364.463	302.923	0.8mm
9	T-16, stake in the flood-land, station of the year 2002. "Ford"	480.362	301.141	2.8mm
10	T-17, wooden pole in the flood-land, station 2002. "Pole Southern"	525.594	344.973	3.2мм
11	TRP-19, metal rod on the meadow in the area of haymaking	109.579	006.258	4.5мм
12	BSS, cliff mark	579.270	330.523	3.1мм

Table 3: Error evaluation for the TRP-19 station location

№ S	Up to the station	m_s, mm	γ, º	m_s^2	$m_s^2/\sin^2\gamma$
22	TRP-10	1.8	43.7	3.24	6.7879
25	T-13	1.8	43.7	3.24	6.7879
$[m_s^2/\sin^2\gamma]$=		13.5758			
$m_{s\ TRP\text{-}19}$=	3.7				

d) Preliminary error evaluation for the TRP-10 station location. The triangular with peaks in points TRP-10, TRP-12 and t-12 gave the best crossing; the crossed ranges were the network sides TRP-19-TRP-10 (N22) and TRP-10-t-12 (N7). The m_{22} and $m_{7\ values}$ were given in the

Table 12 and the $\gamma = 89.7^o$ was measured by using inclinometer. Therefore, $m_{s\ TRP\text{-}10}^2 = (2.1^2 + 1.8^2) / sin^2 89.7 = 7.65$ and $m_{s\ TRP\text{-}10} = 2.8mm < 6mm$ (Table.4).

Table 4: Error evaluation for the TRP-19 station location

№ S	Up to the station	ms, mm	γ, °	m_s^2	$m_s^2/sin^2\gamma$
7	t-12	2.1	89.7	4.41	4.4101
22	TRP-19	1.8	89.7	3.24	3.2401
$[m_s^2/sin^2\gamma]$=		7.6502			
$m_{s\ TRP\text{-}10}$=	2.8				

e) Preliminary error evaluation for the BSS station location. The triangular was taken for cross ranging with peaks t-7, BSS, t-17 and sides t-7–BSS (N29), t-17–BSS (N33). The m_{29} and m_{33} values were given in the Table 12 and the angle $\gamma = 71.6^o$ was measured by using inclinometer. So, $m_{s\ BSS}^2 = (2.1^2 + 1.1^2) / sin^2 71.6 = 6.2$ *u* $m_{s\ BSS} = 2.5mm < 6mm$ (Table 5).

Table 5: Error evaluation for the BSS station location

№ S	Up to the station	ms, mm	γ, °	m_s^2	$m_s^2/sin^2\gamma$
29	т-7	2.1	71.6	4.41	4.8980
33	т-17	1.1	71.6	1.21	1.3439
$[m_s^2/sin^2\gamma]$=		6.2419			
$m_{s\ BSS}$=	2.5				

f) Preliminary error evaluation for the OS-12 station location. The triangular with peaks in points t-14, OS-12, t-15 was used for crossing and sides t-14–OS-12 (N5) and t-15–OS-12 (N6). The m_5 and m_6 values were given in the Table 12 and the angle $\gamma = 95.9^o$ was measured by using inclinometer. So, $m_{s\ BSS}^2 = (0.1^2 + 1.1^2) / sin^2 95.9 = 1.6$ and $m_{s\ BSS} = 1.3mm < 6mm$ (Table 6).

g) In accordance with the preliminary measurement program the double line measurement was planned – one measurement in the one direction and the other one in the opposite direction. This type of measurement was based on the fact that in the course of the second measurement major errors could be detected right away. In fact this was particularly important for the person, who didn't have the required experience and conducted the measurement of the vertical angles by using inclinometer. The preliminary error evaluation in the determination of the four stations (TRP-19, TRP-10, BSS, OS-12) supported the adequacy of such measurement program: 3.7mm, 2.8mm, 2.5mm and 1.3mm to a much lesser extent than 6mm. Therefore, the weights of the network sides were determined based on the calculated measurement errors (Table 7). The value $\mu_d = 2.1mm$ (the error in measurement of the side with a weight of $P = 1$, in our article this was a network side, which could be 100-200m long) was taken as weight unit error.

By using the calculation program version of the strict equalization for the "fluctuating" geodetic network, which was modified for this purpose, the forecast of the limit errors for the relative position of the basic network points was put together. The result showed that the project conformed to the requirements of the project: $MAX(m_s) = m_{TRP\text{-}19} = 5.2mm < 6mm$ *and* $m_{s\ average} = 2.4mm = 2.8mm < 3mm$ (Table 8).

Table 6: Error evaluation for the OS-12 station location

№ S	Up to the station	m_s, mm	γ, °	m_s^2	$m_s^2/\sin^2\gamma$
5	T-14	0.7	95.9	0.49	0.4952
6	T-15	1.1	95.9	1.21	1.2229
$[m_s^2/\sin^2\gamma]$=		1.7181			
$m_{s\ OS\text{-}12}$=	1.3				

Table 7: Errors in measurement of the basic survey network sides (μ_d)

Line	Single measurement	Double measurement
Up to10m	0.5mm	0.5mm
Up to50m	1.0mm	0.7mm
Up to100m	1.5mm	1.1mm
Up to220m	3.0mm	2.1mm
300-700m	2.5mm	1.8mm

Table 8: The forecast of the limit errors for the relative position of the base survey stations m_s

№	Base survey station	m_s	m_s^2
1	T-7 (stake, driven into the ground)	3.3mm	10.98
2	T-8 (wooden rode pole)	1.8mm	3.29
3	T-9 (wooden rode pole)	0.9mm	0.87
4	TRP-10 (stake, driven into the ground)	1.9mm	3.73
5	OS-12 (marking pole for the underground cable)	1.3mm	1.61
6	T-13 (metal sepulchral landmark)	2.7mm	7.36
7	T-14 (wooden rode pole)	1.9mm	3.53
8	T-15 (деревянный дорожный знак)	0.9mm	0.80
9	T-16 (stake, driven into the ground)	3.2mm	10.52
10	T-17 (wooden pole, driven into the ground)	3.8mm	14.19
11	TRP-19 (metal rod, driven into the ground)	5.2mm	27.52
12	ОПП (cliff mark)	3.6mm	12.78
	$m_{s\ average}=([m_s^2]/12)^{1/2}$		*2.8mm*

h) In order to avoid the errors in the course of the centering process, the multi-tripod technique was: the measurements were carried out by using many photo-tripods, where on the alternate capping special Disto target or ranging device Disto Classic was installed. With slope angles of up to 1° and distances of up to 100m, the measurement of the vertical angle was carried out by apposition of the optical inclinometer to the upper surface of the ranging device. Limit errors (reliability level of 95%), caused only by errors in the course of optical inclinometer angle measuring were given in the Table 9.

In case of distances over 100м or slope angles exceeding 1°, the vertical angle measurements were carried out by using an optical theodolite 2Т5К and a ranging device was fixed on its observation glass with a special fastener. Distances over 220m were measured by using electronic tacheometer Pentax II. Measurements by using ranging device Disto Classic were carried out in the forward and reverse directions and in the case of tacheometer Pentax II only in one direction, but twice. Also, deviation between the measured distance values has been under control: no more than 5mm (for the lines > 100m), 2mm (for lines 50-100m) and 1mm (for lines <50m). RMS errors of a single line measurement by using ranging device Disto Classic constitute 1.5mm per 100m, by using electronic tacheometer Pentax II – 2.5mm for lines of 600m level (Table 9). S-measured slant ranges were brought up to the horizon by means of corrections, and corrections were entered into the horizontal range of the network sides S_α in order to bring it to the sea level δ_H and to Hauss-Krüger δ_Y projected scale.

Table 9: Errors in line measuring in (mm), caused only by the error in measurement of the inclination angle (by using optical inclinometer 0.1º), considering small distances and inclination angles

	20m	**40m**	**60m**	**80m**	**100m**	**120m**	**140m**	**160m**	**180m**	**200m**	**220m**
0.5º	0.3	0.7	1.0	1.3	1.7	2.0	2.3	2.7	3.0	3.4	3.7
1.0º	0.6	1.3	1.9	2.6	3.2	3.8	4.5	5.1	5.8	6.4	7.0
1.5º	0.9	1.9	2.8	3.8	4.7	5.7	6.6	7.6	8.5	9.4	10
2.0º	1.2	2.5	3.7	5.0	6.2	7.5	8.7	10	11	12	14
2.5º	1.6	3.1	4.7	6.2	7.8	9.3	11	12	14	16	17
3.0º	1.9	3.7	5.6	7.4	9.3	11	13	15	17	19	20
3.5º	2.2	4.3	6.5	8.6	11	13	15	17	20	22	24
4.0º	2.5	4.9	7.4	9.9	12	15	17	20	22	25	27
4.5º	2.8	5.5	8.3	11	14	17	19	22	25	28	30
5.0º	3.1	6.1	9.2	12	15	18	22	25	27	31	34

i) Measurement data treatment included pre-processing (initial treatment – part of field work) and post-processing (equalization and accuracy evaluation - part of office calculation).

- The initial data treatment was conducted in field conditions right after field work completion and in this case included control over all necessary corrections in distance data and getting length values for the ranging sides of the network. In our case, the initial treatment included all necessary distance corrections in order to get the length values for the network sides on the plane of the six-degree zone №45 of Hauss-Kruger projection (SK-42) and to calculate the approximate network station coordinates. The approximate coordinates of all stations were calculated in accordance with linear marks with the best geometry and the base coordinates were the ones, obtained from the satellite survey (with consideration of the State Survey Network station data). The programs for calculation of the required corrections and the solution of cross ranging have was carried out by using calculator "Casio Algebra 2.0+" in order to avoid calculation errors and to save the time. Major measurement errors were controlled in the field by using these programs and a calculator, considering its energy-independent flash-memory was used as a storage and "imager" of information.

- Adjustment and evaluation of accuracy was conducted by using "Casio Algebra 2.0+".

The algorithm of strict adjustment of the "fluctuating" survey network was applied. The program automatically selected the basic data from the electronic field diary, calculated the coefficients of the correction equations for the measured network sides, solved the system of normal equations by applying pseudo inverse matrix and gave the corrections to measurements (network sides) and parameters (network points' coordinates). On the last stage of calculation the accuracy evaluation for the adjustment of coordinate values and network side lengths was carried out based on the reversed weights matrix. Prior to the start of the current year's material processing, the last year's basic network strict adjustment was carried out. The exact limit error for the station locations in 2002 wasn't predetermined. The results of the base survey station processing for 2002 were given in the Tables 10, 11.

Due to the fact that the measurements in the basic survey network were uniform, the weights assigned to them were the all the same. Reliability of the accuracy evaluation considering such low number of the excessive measurements was very low, but it was enough to have even the estimated values for the relative position of the basic survey network points (for year 2002) in order to conclude, that the last years' measurement data weren't acceptable for the purposes of the project for the year of 2003. Final adjustment results for the project 2003 were given in tables 2 and 12. Man-hours constituted 1 week (the team consisted of supervisor and rodman) and required serious preparation in geodesy for the work team.

Table 10: Base survey network parameters (year 2002)

Number of measurements	14
Number of the unknowns	12
Excessive measurements	2
RMS error for the unit of weight, μ	3.23 mm
Average error of the relative points' position, $M_{average}$	7.31 mm

Table 11: Relative position errors for the base survey network points (year 2002), obtained in the course of processing in accordance with algorithms of strict adjustment by using the parameter method

№ and the name of a base survey station	**Ms**
1 "Reference point"	8.25 mm
2 "Pole Northern"	5.74 mm
3 "Pole Southern"	6.19 mm
4 "Ford"	8.40 mm
5 "Eastern ablution"	7.57 mm
6 "Western ablution"	7.28 mm

3.3.5 Summary

In accordance with the modern approach to the accuracy enhancement, the general location of the archaeological site can be conducted inaccurately, which only allows to locate it and to place it on the topographic map or the plan of the required scale.

Table 12: Corrections to length values of the measured distances, obtained from the basic survey network

№	Line №1→№2		Length in SK42	Measurement error	Line weight	Correction from equalization
1	3	12	103.828 m	2.1 mm	1.00	0.99 mm
2	3	14	64.391 m	1.1 mm	4.00	0.33 mm
3	3	15	141.291 m	2.1 mm	1.00	0.27 mm
4	4	15	76.900 m	1.1 mm	4.00	-0.22 mm
5	2	14	48.086 m	0.7 mm	9.00	0.16 mm
6	2	15	55.253 m	1.1 mm	4.00	-0.01 mm
7	10	12	190.347 m	2.1 mm	1.00	-0.51 mm
8	10	15	167.817 m	2.1 mm	1.00	0.28 mm
9	10	9	124.691 m	2.1 mm	1.00	0.37 mm
10	9	12	71.774 m	1.1 mm	4.00	0.26 mm
11	9	15	45.245 m	0.7 mm	9.00	-0.10 mm
12	9	8	117.263 m	2.1 mm	1.00	1.00 mm
13	16	8	183.549 m	2.1 mm	1.00	-0.85 mm
14	16	BSS	103.171 m	2.1 mm	1.00	0.86 mm
15	16	9	128.147 m	2.1 mm	1.00	0.67 mm
16	16	15	115.906 m	2.1 mm	1.00	-0.44mm
17	8	12	53.681 m	1.1 mm	4.00	-0.46 mm
18	8	14	6.154 m	0.5 mm	36.00	0.00 mm
19	8	13	58.630 m	1.1 mm	4.00	-0.58 mm
20	14	9	111.109 m	2.1 mm	1.00	-0.74 mm
21	16	14	179.237 m	2.1 mm	1.00	1.55 mm
22	19	10	310.288 m	1.8 mm	1.44	-0.24 mm
23	19	9	354.819 m	1.8 mm	1.44	0.09 mm
24	19	12	363.083 m	1.8 mm	1.44	0.17 mm
25	19	13	425.846 m	1.8 mm	1.44	-0.31 mm
26	19	15	391.112 m	1.8 mm	1.44	0.20 mm
27	7	10	163.312 m	2.1 mm	1.00	-0.51 mm
28	7	16	73.058 m	1.1 mm	4.00	0.04 mm
29	7	BSS	109.822 m	2.1 mm	1.00	-0.56 mm
30	19	BSS	570.724 m	1.8 mm	1.44	0.06 mm
31	17	16	62.984 m	1.1 mm	4.00	-0.02 mm
32	17	8	218.803 m	2.1 mm	1.00	-1.14 mm
33	17	BSS	55.562 m	1.1 mm	4.00	-0.15 mm
34	17	9	187.325 m	2.1 mm	1.00	0.34 mm
35	17	15	166.521 m	2.1mm	1.00	0.00 mm
36	17	14	215.774 m	2.1 mm	1.00	0.30 mm

In this case, the ranging results of the excavation execution survey have to be conducted with a high level of accuracy (in respect to the 3D physical coordinate network fixed at the excavation site) [Gulyaev, Vasiliev, 2001]. The accuracy of the 3D physical coordinate network has to be optimally coordinated with the excavation survey accuracy. By analogy with the networks in construction, the design of the accuracy parameters set for the overall archeological-geodetic project can be not directly correlated with the set of accuracy parameters for the excavation network.

Geodetic research in 2003 was devoted to the search of the optimal techniques in field and lab work.

a) Field and lab work

- satellite positioning (absolute values for coordinates) of the base survey stations at the excavation sites in the required frame of reference (locating of site, coordinate values can have errors up to several meters) and

- ground-based instrumental measurements (micro-trilateration) for obtaining object coordinates at the excavation area relative to the base survey stations with limit error, which satisfies requirements of "Archaeological excavations and prospecting regulations and licensing" (millimeters – for the relative position of the base survey stations, centimeters – for errors in location of other objects relative to the base stations).

b) Satellite observations

Conditional accuracy of the satellite positioning is accuracy, applicable in case of certain navigational system work conditions, used techniques and observer qualifications. The conditional accuracy (*RMS error*) of satellite positioning, produced by one code GPS-receiver with $HDOP = 4$, is equal to $m_{HDOP=4} \sim 2\text{-}2.5м$ in relation to base survey stations. Considering the high-quality preliminary forecasting of satellite observation work sessions it is possible to select for the observation sessions the time period with minimal value of the HDOP-factor. This allows obtaining the coordinate increments with *RMS error* less, than 1.5m. In order to preserve the integrity in land measuring by using navigational GPS-receivers in satellite positioning not only conditional accuracy, but also measurement integrity maintenance is required. The system must be capable of providing the users with timely warnings in cases, when the system can’t be used in solving of the required geodetic task. The integrity of the completed satellite positioning is achieved by the way of joint processing of the satellite observations, which are executed at the points of a single net, consisting of the determinate points, source and base points in accordance with the prepared measurement program. The equalization and calculation of the determinate points’ coordinates is carried out by using software, which has been developed by E. Vergounov for joint processing of the satellite relative measurements network. Analysis, equalization and accuracy evaluation for the relative satellite positioning are conducted as for the "fluctuating" geodetic network. The lab processing consists of the following several stages:

- Field recording from the memory of receivers of fixation coordinates, considering local corrections to the orientation of all-earth ellipsoid, numeration of the computation lines and selection of base lines.

- Filling of the data matrix in the engineering calculator.

- Strict network adjustment is carried out by using basic points with known coordinates in the required frame of reference (parametrical method of smallest squares). Base lines are used as parameters. After the preliminary network equalization, in accordance with the data filtration principle of "statistical funnel", all computation lines with "major errors" are excluded. After this the operator makes a decision about which one of the computation lines to use in the future processing, looking at each of the groups of repeated computation lines. If the coordinates of the line ends in the course of the repeated measurements differ by no more than 1m, the average value of these measurements is taken as a final one.

- Conducting the evaluation of precision for the adjusted coordinate values.

c) Ground-based instrumental measurements

- Creation of geodetic networks (micro-trilateration areas, consisting of point network with distances of tens of meters between the points) by using the set of instruments for ranging measurements.
- The analysis, equalization and precision evaluation of the instrumental geodetic network. This geodetic network is treated as "fluctuating" network of micro-trilateration. The coordinates of the base stations of this network, obtained from satellite network equalization, are taken as equalization parameters. All BSS, TRP, OS are unified into a single geodetic network of the points with an average error of the relative points' position equal to $\mu_{s\ average} = 2.4mm$ by applying direct ranging measurements between BSS, TRP, OS and connecting points.
- Projection into the flood-land of Charysh River valley of the Ust-Kanskaya Cave excavation coordinate network for determination of azimuths of its sides by using the base points of the micro-trilateration network.
- Obtaining of the absolute elevation marks of the zero level for the Ust-Kanskaya cave excavation through trigonometric leveling by using electronic tacheometer.

At the end of the field season of 2003 it became clear that application and use of the mobile instrument set, which includes:
- one or two GPS-navigators ("Garmin", USA),
- "laser tape measuring device" (Laser ranger Disto classic[5] "Leica", Switzerland) with targets and tripods (Russia),
- optical inclinometer, combined with survey compass (Tandem "Suunto", Финляндия),
- engineering programmed graphical calculator ("Casio", Japan),

- laser level (Limka "Laser devices", Russia),
- chronometer, which works on solar batteries with digital compass, barometer and barometric altimeter (Protrek "Casio", Japan)

is appropriate for the purposes of the field archeological research. The mobile instrument set can be easily transported to the working area by one person, no transportation means are necessary to transport the set. There are no customs restrictions pertaining to transportation of this instrument set and there are no special requirements from the local authorities for the use of the set in the work area. The cost of this instrument set is significantly lower, than the cost of the modern tacheometer. Independence of transportation means and energy sources can facilitate the autonomy of operation for the field season by using the ordinary batteries.

At the same time there is an issue of accuracy level in this work: humanity specialists usually don't have adequate qualification to carry out work of such high precision. It seems, that in order to fixate the work area (to get the absolute coordinates for the excavation base survey stations or excavation system) the scales of 1:25000–1:100000 are adequate for the archaeological research. Fixation by using navigational satellite receivers is justified in this case. The system of theodolite computation line in mountainous area (elevation differences of 460-1000m) and long distances to the SGN points (7-9km along the straight path) are much more complicated and less accurate. In fact, position RMS error for the points, taken as SGN points in case of our navigational satellite determinations is equal to 0.3m. RMS error for the points, the coordinates of which have been used as parameters in the course of the ground-based instrumental coordination equalization is in the range from 60 to150 cm ($\mu_{s\ average} = 1.2m$).

So, $M_s = (2.4^2 + 0.6^2)^{1/2} \sim 2.5m$ can be taken as limit position error of the satellite coordinate determinations for the points interesting to us. The accuracy of placing of the points onto the topographical map of the 1:25000 (the largest scale of the topographic map for the work area) without taking into consideration paper deformation and polygraphic defects constitutes:

- when using cartometry method for determination of the exact contours of the separate objects – $m_s = 12.5m$ *and* $M_s = 25.0m$, and "fluctuating" contours and relief features – $m_s = 25m$ and over. The error of the absolute elevation mark can reach the cross-section height of the map;
- when using plan coordinates for SGN stations, taken out of the catalogue – $m_s = 2.5m$, $M_s = 5.0m$, obtained by using the cartometry method – $m_s = 7.5m$, $M_s = 15.0m$.

Due to the fact that precision of our satellite positioning of the support points doesn't exceed $m_s = 4.6m$, the method of satellite navigational positioning is adequate for any planned geodetic work, considering the scale accuracy of 1:25000. Besides, after the analysis of the navigational satellite measurements, it was recommended to "shift" the image by dX=-12m (12 m to the west) and dY=12m (12 m to the north) for better correlation with the area. Considering the fact, that the map was put together based on the materials of 1950s (it was renewed in 1981), the geodetic work at the Ust-Kanskaya cave made it possible to update the section of the topographic map. The accuracy of the work conducted is more than adequate to its scale.

4. FINAL SUMMARY

In the course of this research it has been determined, that:

- for application of geodesy in the archaeological research independent division has to be defined – archaeological geodesy;
- the parameters individual for each site (excavation structural unit) help to facilitate accuracy control in geodetic support. These parameters are: the sampling area (minimal size excavation area) and sampling thickness (sampling depth).

Based on the conducted research new effective methods of geodetic support have been developed for the 3D physical survey network. Comparative evaluation of the developed techniques has demonstrated the effectiveness of the introduced methodology, based on the combined application of the satellite (navigational) positioning methods (including determination and account of SGN network deformations in working area) and ground-based instrumental (accurate) geodetic measurements.

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