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The Kislaya Guba tidal power plant

by L. B. Bernshtein, V. G. Gavrilov,
S. L. Gel'fer, N. N. Nehoroshev, L. I. Suponitskii,
I. N. Usachev, M. L. Monosov, V. N. Silakov, I. M.
Pylev, V. I. Platov, V. L. Vestfal, and M. S. Trifel

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Dear Mr. G. Godin

We are sending to you this monograph on the Kislaya Guba experimental tidal power plant as a sign of our gratitude for the material on the Canadian tidal power plant projects which you kindly sent to us.

Sincerely yours,

(signed)
A. Stolyarov

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THE KISLAYA GUBA TIDAL POWER PLANT

/2*/

L.B. Bernshtein

General Editor

Authors: L.B. Bernshtein, V.G. Gavrilov, S.L. Gel'fer, N.N. Nehoroshev,
L.I. Suponitskii, I.N. Usachev, M.L. Monosov, V.N. Silakov,
I.M. Pylev, V.I. Platov, V.L. Vestfal, M.S. Trifel

This book examines the experience of designing, constructing, and testing the experimental Kislaya Guba tidal power plant, which was created in order to examine new methods of utilizing tidal energy, accomplished for the first time in the practice of hydroelectric power plant construction by the floating block method.

The new simplified design of the structure, its computation, new materials (especially frost-resistant and waterproof concrete, epoxy resin foam hydrothermal insulation, hydrophobic soil) and the methods of shielding them which have been developed and applied during the construction of the Kislaya Guba tidal power plant have an important significance for marine and hydraulic engineering, which results in this book being of interest to a wide circle of specialists in various fields of hydraulic engineering, instructors, and students at higher institutes of engineering.

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Preface

This book is a result of research into the problem of utilizing tidal energy which the All-Union Planning, Surveying and Scientific Research Institute has been conducting for 30 years. The solution to this problem has turned out to be very complex, as was stated by the Soviet scientists Yu.M. Shokal'skii, V.V. Shuleikin, and V.E. Lyakhnitskii in their papers as early as the 1920's. This is attested to by the numerous foreign tidal power plant projects which had not been realized as of 1966.

The need for new "clean" sources of energy, however, has necessitated a search for ways of efficiently utilizing the energy resources of the tides and has led to the construction of an economically significant tidal power plant on the Rance River in France. Unfortunately this installation, which revealed the power production capability of the tide, was not able to resolve the problem of the economic soundness of tidal power plants.

The Kislaya Guba tidal power plant indicates the way to such economic soundness. Its planned capacity of 800 kW is very small (only one 400 kW unit has been installed so far), but it demonstrates the possibility of a basic decrease in the cost of tidal power plant construction through the use of the floating block method, which is especially efficient under conditions in the Soviet Union where channels with high amplitude tides are far from centers of population and are under severe climatic conditions.

The use of the floating block method during the construction of the Kislaya Guba tidal power plant required the solution of a series of complex engineering problems: the construction of a new skeleton structure, materials for building it, their shielding, construction of the underwater foundation, use of the reversible bulb unit, and creation of a generator with variable speed of rotation. All of the above is also important at

this time in various areas of hydraulic engineering and construction, and for this reason the exposition of these questions in this book will clearly be of interest to a wide circle of specialists.

The creation of the Kislaya Guba tidal power plant project by Hidroproekt (All-Union Planning, Surveying and Scientific Research Institute) and its realization in Sevgidrostoi (Administration for the Construction and Installation of Hydroelectric Power Plants in the North) was under the direction of Candidate of Technical Sciences L.B. Bernshtein, the chief project and construction engineer. By carefully studying earlier projects, from 1938 on, he determined the reasons for their lack of success, uncovered the positive qualities of tidal energy and methods of realizing them, and also proposed a cadastral method for evaluating tidal energy resources, the design of tidal power plants and the floating block method of constructing the structure. L.B. Bernshtein's monograph "Prilivnye elektrostantsii v sovremennoi energetike" (Tidal Power Plants in Modern Power Engineering) published in 1961, which was a result of his research, won recognition in the USSR and abroad and became the basis for a new approach to the utilization of tidal energy which was realized in the construction of the Kislaya Guba tidal power plant. Understandably this practical stage of work required the creative participation of a number of institutes and construction organizations. The purposeful direction of their creative research within the framework of the coordinated scientific plans and the author's integrated direction of the research, planning and construction assured the successful and timely solution to the above mentioned engineering problems and the entire experiment. That is why this book, which covers the creation of the Kislaya Guba tidal power plant, which realizes the ideas presented in the book published in 1961, is its logical continuation.

D.M. Yurinov, Head
Hidroproekt Institute

From the Editor

The book "The Kislaya Guba Tidal Power Plant" was written by a group of specialists from the All-Union Order of Lenin Planning and Surveying and Scientific Research Institute of Gidroproekt and from other Institutes which took part in the creation of the Kislaya Guba tidal power plant: L.B. Bernshtein (Chapter 1, sections 2-3, 3-1, 4-1,a; 4-2,b, chapters 8 and 12); M.L. Monosov (sections 2-2, 2-3); S.L. Gel'fer (sections 3-2, 3-3, 10-1); V.N. Silakov (section 4-1,b); I.M. Pylev (section 4-2,a); V.I. Platov (sections 4-2,b, 4-3); V.L. Vestfal (section 4-4); L.I. Suponitskii (chapter 5); I.N. Usachev (chapters 6 and 9); M.S. Trifel and N.N. Nekhoroshev jointly (sections 7-1, 7-2), M.M. Nekhoroshev (section 7-3); V.G. Gavrilov (chapter 10, except for section 10-1); L.B. Bernshtein, S.L. Gel'fer and N.N. Nekhoroshev jointly (chapter 11).

In writing this book the authors were guided by the research carried out by Gidroproekt and other Institutes taking part in the realization of the coordinated plans on the problem of utilizing tidal energy, in particular, All-Unions Scientific Research Institute of Transportation Construction (Doctors of Technical Sciences F.M. Ivanov and V.S. Luk'yanov, Candidates of Technical Sciences I.I. Denisov, Yu.A. Krostelev); Scientific Research Department of Gidroproekt (Doctor of Technical Sciences V.M. Lyatkher, Engineer Yu.I. Braslavskii, Candidate of Technical Sciences V.M. Klabukov, Engineers S.G. Dmitriev, P.A. Pshenitsyn, Candidates of Technical Sciences V.L. Sakharov and L.A. Igonin, Engineers M.S. Segal, V.A. Sirotkin, Candidate of Technical Sciences A.I. Tsarev, Engineer M.A. Burmistrov and others); All-Union Scientific Research Institute of Hydraulic Engineering (Candidates of Technical Sciences E.A. Lubochkov, V.I. Sinotin, V.A. Solnyshkov, S.M. Aleinikov, Engineer V.G. Zhebrovskaya and others); State Planning, Design

/5/

and Scientific Research Institute of Marine Transportation of the Ministry of the Maritime Fleet, USSR (Engineers G.V. Tankhel'son, G.V. Khukhrin, I.F. Aleksandrov and others); The Murmansk Main Administration of the Hydrometeorological Service (Engineers V.T. Zhevnovaty, E.A. Sopchenko and others); State Research and Planning Institute LKP (Engineers Yu.B. Shleomenzon, E.S. Gurevich); Leningrad Branch of the State Institute of Oceanography (Doctor of Technical Sciences I.L. Davidan, Candidate of Technical Sciences V.M. Al'tshuler, Engineer N.S. Uralov and others); Kola Regional Administration of Power System Management (Engineers N.I. Zarkhi, B.A. Antonov, R.A. Sirota, V.R. Gorbatenko, N.I. Goverdovskii and O.N. Tyul'kin); State Scientific Research Institute of Building Physics (Candidate of Technical Sciences L.F. Yankelev, Engineer E.V. Fetisov and others); Central Scientific Research Institute of Structural Parts (Candidates of Technical Sciences E.N. Kisyuk, N.S. Ryabov and others); Moscow Construction Engineering Institute (Doctors of Technical Sciences M.M. Grishin, S.M. Slisskii, G.D. Petrov, G.L. Khesin and Candidate of Technical Sciences V.N. Sevast'yanov and others), Leningrad Branch of the All-Union Planning, Surveying and Scientific Research Institute. (M.L. Monosov, N.V. Pavlikhin, V.M. Sadkov, N.S. Fedotov, G.M. Ogurtsov, Yu.V. Petrov, N.N. Simakin, A.V. Trubacheva, N.G. Borshchevskii, S.E. Shmidt, S.N. Osolodkin, E.A. Semenov and others).

Please address remarks and suggestions to: Moscow, M-114, Shlyuzovaya Naberezhnaya, Bldg 10, "Energiya" Publishers.

Chapter 1

THE KISLAYA GUBA TIDAL POWER PLANT AND THE PROBLEM OF UTILIZING TIDAL ENERGY

/7/

1-1. Present Situation

At the end of 1968 the start-up of the experimental Kislaya Guba tidal power plant (TPP) was announced (Fig. 1-1). Why did this small installation with a design capacity of only 800 kW (400 kW installed so far), built 2 years after the French Rance tidal power plant with a capacity of 240 thousand kW, become the object of attention not only of Soviet but also of foreign experts?

To answer this question it is necessary to recall the developmental stages of this problem. The engineering intellect has for many years struggled to harness the energy of the tides. This struggle has had its peaks and slumps, at times assuming a dramatic character, often absorbing the entire life of scientists and engineers dedicated to it (D. Cooper in the USA and L. Defour in France).

Probably no other modern, power engineering problem has provoked such bitter attacks as the design and construction of tidal power stations.

The older generation of engineers probably remembers what a storm of indignation engulfed Cooper's project in 1935 for the Passamaquoddy (Quoddy) TPP which was begun in the bay of this name on the border between the USA and Canada. H. Riggs, the Chairman of the Society of Civil Engineers at that time, called this project "economic folly" while W. Carpenter, an economist at the Edison Institute dubbed it an "expensive whim". Construction, which had hardly begun but had, it is true, swallowed up six million dollars, was stopped after six months. An objective examination of the indices of the Quoddy project shows that the abandonment of construction was justified. Indeed, it cannot be considered economically sound to

construct an electric power plant at which, in order to obtain a guaranteed capacity of 30,000 kW, it is necessary to install 280,000 kW (tidal power plant + pumped-storage electric power plant), to construct huge dams with gates, embankments, in short, to spend two thousand dollars per 1 kW of guaranteed capacity and to obtain energy at 1 cent per 1 kW hour, when it is possible to build hydroelectric power plants on the rivers of that area, the energy from which would cost 0.187 cents per 1 kW hour with an outlay of 550 dollars per 1 kW.

But now tidal power, which appeared to have been buried in 1935 on the border between the American state of Maine and the Canadian Province of New Brunswick, has been reborn 32 years later in the form of the Rance TPP, with a capacity of 240,000 kW.

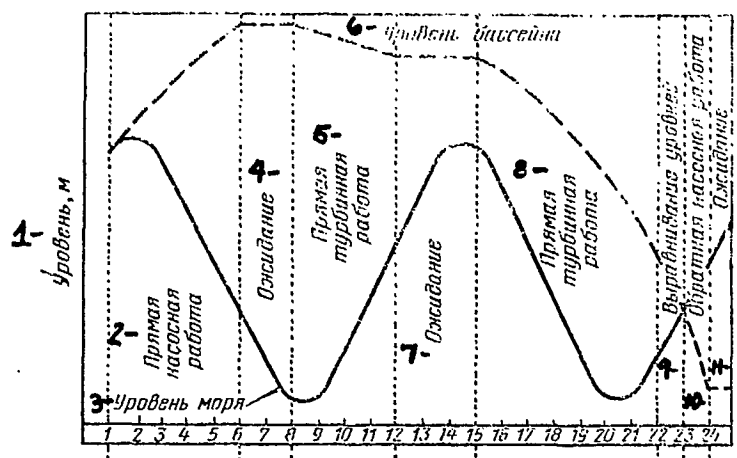


Figure 1-1. General view of the Kislaya Guba TPP.

1 - Level, meters; 2 - Direct pumping operation; 3 - Sea level; 4 - Wait; 5 - Direct turbine operation; 6 - Basin level; 7 - Wait; 8 - Direct turbine operation; 9 - Equalization of levels; 10 - Reverse pumping operation; 11 - Wait

When he opened this power plant General DeGaulle, the President of France, called it "the outstanding installation of the century", but at the same time R. Marcellin, the Minister of Industry, stated that in the future the construction of atomic power plants would have priority over the construction of the super-powerful Chausey tidal power plant (12 million kW). For this reason the newspaper Humanité carried the news about the start-up of the Rance TPP under the heading: "Triumph or Funeral".

It appeared that tidal power was dethroned again. And yet after Rance the Soviet experimental Kislaya Guba TPP was built and led to the creation of a series of projects for powerful tidal power plants; in the USSR, England (Severn), Canada (Fundy) and the USA (Quoddy once again). /9/

It is very tempting to harness a force which rhythmically raises the water of the oceans by 5-10 meters, slows down the rotation of the earth, and whose potential is evaluated at 1 billion kW.

The current interest in tidal energy is a result of its seasonal and annual constancy and the absence of atmospheric pollution, damage to the fishing industry, or flooding during its utilization and also as the result of the almost total assimilation of "clean" river energy resources.

The main obstacle to the utilization of tidal energy has until recently been the variable nature of the diurnal range of the tide and the rhythm of these oscillations occurring in lunar time (i.e. the daily 50 minute displacement of the tides), as well as variation in the height of the tide over a fortnightly period following the phases of the moon. Scientists tried for decades to solve the problem of transforming this pulsating energy into a constant capacity suitable for utilization. The latest French and Soviet research has shown, however, that under the conditions of our present economy, in which the consumption of energy has a

wave-like nature determined by the rhythm of man's life, it is not necessary to strive to smooth out the pulses of tidal energy which follow lunar time. The problem consists simply in integrating these pulses with the "waves" of consumption which follow solar time. In such a case tidal energy, whose mean monthly magnitude is constant regardless of the annual and seasonal rainfall, becomes an extremely important and valuable component of the power grids which unite hydraulic and thermal (coal and atomic) electric power plants.

The problem of matching the tidal energy cycle with the demand cycle is solved with the aid of the pump effect of a reversible, axial hydroturbine unit.

In essence, the pump effect consists in pumping water into the TPP basin above the high-water level or pumping water out to below the low-water level. When this is done (Fig. 1-2) the energy of the tide is stored and may be utilized at any time regardless of the phase of the tide. Due to the fact that pumping water in (or out) must be carried out at times close to high (or low) water, the efficiency factor of a TPP, as opposed to a pumped storage electric power plant, turns out to be greater than 100% /10/ because the stored sea water is discharged through the turbines at a higher head than when it was pumped in (as the result of lower sea level during ebb or higher sea level during flood). The idea of utilizing the reversibility of the adjustable-blade turbine for achieving a pump effect of TPP's was proposed in 1947 by L.B. Bernshtein [12]. The creation of reversible bulb-type units with a horizontal axis of rotation, operating with a high efficiency factor under six regimes (forward and reverse turbine and pump operation, forward and reverse throughput) provides for the flexible utilization of the TPP not only to handle peak loads but also to use the free

capacity of thermal electric power plants during hours when they are not handling their full load. During such hours the TPP assembly becomes a pump and, absorbing the free energy of the system, pumps seawater into the TPP basin above the high-tide mark (or pumps water from the TPP basin into the sea to below the low-tide mark).

It appears to be encouragingly simple to use for this purpose an axial hydroturbine with an inclined axis and a durable generator of the type manufactured in the USA by "Allis Chalmers" for river hydroelectric power plants and proposed for the Passamaquoddy tidal power plant [6], or the direct current hydroturbines proposed in the most recent design for the Severn tidal power plant in England [56]. In this case the TPP is built in accordance with the double-basin scheme [3], which ensures either continuous operation or operation during the half-peak part of the curve with storage of the free capacity of the thermal electric power plant during the hours of the nightly drop in consumption (see the Mezen tidal power plant scheme in Fig. 1-4). The double-basin TPP scheme, however, permits only 50% of the potential energy of the basin to be utilized. The most complete utilization of potential is possible with a single-basin scheme which permits the operations of a TPP in the sharp part of the curve. A tidal hydroturbine with bulb-type units is required for this type of scheme (section 4-2).

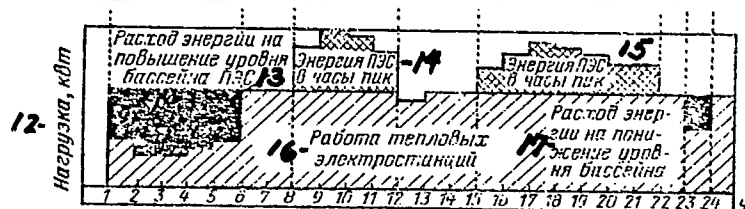


Figure 1-2. Model graph of the operation of a single-basin TPP under peak regime.

12 - Load, kW; 13 - Energy expended on raising the TPP basin level; 14 - TPP power during peak hours; 15 - TPP power during peak hours; 16 - Work of thermal power plants; 17 - Energy expended on lowering the basin level.

As a result of its extremely favorable hydraulic contours (small encapsulated generator) and horizontal orientation, this unit permits maximum utilization (efficiency factor up to 91%) of the tidal energy during direct turbine operation which takes place, for example, at the Rance tidal power plant during 80% of the plant's operating time.

The unit operates under a 6-cycles regime: 2-way operation as a turbine, pump and water throughput aperture. The reverse regimes are accomplished at a relatively low efficiency factor but it is precisely their combination with the other regimes which makes possible the operation of the TPP during peak hours regardless of the phase of the tide, or during a fuel economy regime at the thermal electric power plants operating in conjunction with the TPP, i.e. at the base of the load graph.

The operation of a TPP under a regime of covering peak loads is given in Fig. 2, which shows the operating cycle of a single-basin TPP. Let us imagine that at 1 AM the grid load (lower graph) has dropped off and the thermal electric power plants are operating at partial capacity. At such time the TPP turbines may be run directly as pumps (upper graph). The generator becomes a motor; using the energy of the idle thermal electric power plants it begins to pump water from the sea into the TPP basin. By 6 a.m. the pumping up of the basin is finished and the TPP switches over to a standby phase. At 8 a.m., when the capacity of the thermal electric power plants becomes insufficient to handle the growing loads, the TPP hydro assemblies are switched to direct turbine operation from the basin into the sea and feed energy into the grid until the moment the peak begins to drop, i.e. until the beginning of the lunch hour, when the thermal power plants alone satisfy the energy requirements. Later, after another standby phase, and with increasing load, the TPP turbines begin to operate. At night they can switch to reverse pumping operation or remain in the standby phase.

The Rance tidal power plant, whose 24 turbines produce almost their full capacity at the peak of the unified energy grid graph, are a shining solution to a significant part of this problem.

This solution was achieved as the result of the realization of R. Gibrat's idea [26], whose outstanding work made it possible to create the Rance TPP project and to unite for its realization the genius of French scientists, engineers, and workers. The comprehensive research which was carried out over the course of almost a quarter of a century had a tremendous significance not only for the creation and realization of the Rance TPP project but also for the solution of the entire problem [52]. Thus, the group of scientists which included L. Vantroys, R. Allard, and Gougenheim, examined the questions of the power engineering and dynamics of the tidal wave under natural and regulated conditions; M. Penel and F. Voyer worked out methods for calculating TPP regimes on the bases of Gibrat's theory of cycles [26]; this collective of SEUM (Research Society for Tidal Power Utilization) engineers which also included R. Auroy, G. Manboussin and R. Sui, planned and actually built a full-scale TPP structure of original design. We should also note the very great contribution of Academician A. Caquot who proposed and validated dam construction using floating caissons the space between which is closed off at slack water before ebb, and the work of engineers R. Rat and G. Surelle, who carried out research on the corrosion of concrete and metal in seawater and who discovered and applied effective measures against corrosion applicable to the conditions at the Rance TPP (cathodic protection together with special polyvinyl chloride coatings and concrete mixtures).

But the most important research was the design by a group of engineers (L. Camerloache, S. Cassaci, A. Ruelle, G. Delpeche and others) of a reversible bulb-type unit, which made it possible to realize Gibrat's cycles.

Thus, as exemplified by the Rance TPP, which has been operating under peak regime since 1967, the possibility of a practicle solution to the first part of the problem (integration of the tidal cycle with the demand cycle) has been proven.

How then can we explain the above mentioned decision of the French government about the priority of building atomic electric power plants before carrying out the Chausey TPP project? The answer to this question is not provided in either R. Marcellin's quoted speech or in subsequent publications in the French technical press. In our opinion there are two causes retarding the construction of the Chausey TPP: the first is the need for intramonthly ("intersyzygial") regulation of the TPP, and the second is the need to significantly lower the cost of building tidal power plants.

Let us examine these circumstances in greater detail.

French and Soviet research has shown that electric tidal power stations are not in conflict with but may, on the contrary, be in harmonious association with atomic power plants. This was mathematically proven by R. Gibrat [26]. The effectiveness of such a tidal-thermal (atomic) power plant association has been shown in examples proposed in the works of L.B. Bernshtein [3] for a number of countries whose power engineering and natural conditions permit the utilization of tidal energy.

It is completely obvious that if tidal power plants could assume some significant portion of the peak loads, then existing and future super powerful thermal (including atomic) electric power plants could operate smoothly in the base portion of the load curve. Technical difficulties when atomic power plants are taking part in peak regime operation are not understood to be part of "smooth operation". It is known that these difficulties have already been successfully overcome. In mind under this aspect is the effectiveness of operation of super powerful tidal (and atomic) power plants

in base regime as determined by their energy cost structure.

The chief determining role in the fixed part of the costs of modern TPPs is played by the expensive rapidly amortizing equipment. The magnitude of these costs does not depend on the number of hours of operation of a TPP (APP) per year. It is thus clear that such installations must operate the maximum number of hours per year in order for each kW hour produced by them to bear the minimum amortization deduction. Consequently, the thermal (atomic) - tidal - hydroelectric power plant system, in which the member tidal and hydroelectric power plants could carry the entire peak portion of the load curve, makes it possible to significantly decrease the cost of tidal and hydroelectric power plant energy by increasing the number of hours of utilization of thermal power plants. In order to arrive at such a harmonious system, however, it is necessary to ensure for the TPP not only the synchronization of the tidal energy cycle with the demand cycle (intradaily regulation) but also compensation for the decrease in the tidal potential over a weekly period which follows the phases of the moon from syzygy to quadrature (so called intersyzygial intramonthly regulation). This problem may be effectively solved by greater utilization of the reservoirs of river hydroelectric power plants which are seasonally regulated.

An example of such a solution, as proposed in the above mentioned research [3] is the scheme of using the energy of super powerful TPPs on the coast of the Cotentin Peninsula in a unified system of the energy grids of England, France, Norway, and Sweden (fig. 1-3). It is proposed in this scheme that there be a tidal power plant at Cotentin, with a capacity of 33 million kW, which should operate in a system including hydroelectric plants with reservoirs which would ensure the possibility of intramonthly (intersyzygial) compensated regulation. In periods of the highest (syzygial) tides the jointly operating river hydroelectric power plants would decrease

their capacity as a result of the increase in the capacity of the TPP, thus saving water; during the period of the highest tides (at quadrature) this water would be used to increase the capacity of the hydroelectric power plants, thus compensating for the decreased capacity of the TPPs.

Clearly, in order to keep the water stored during syzygy in the reservoirs of river hydroelectric power plants a certain additional volume is required and their capacity must be increased in order to utilize the stored water.

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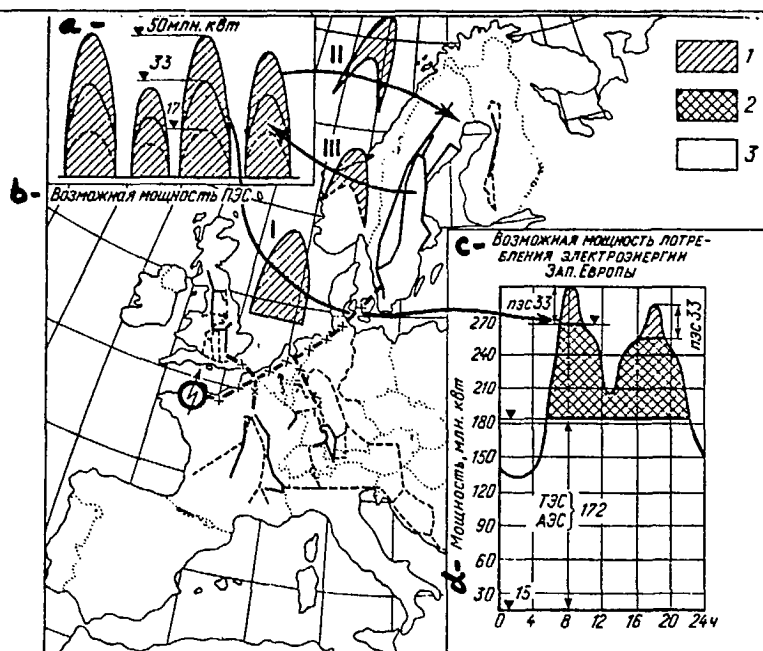


Рис. 1-3. Схема обмена энергией между речными и приливными электростанциями на примере ПЭС Котантен [Л. 3].

— — — существующие «островные» объединения энергосистем Западной Европы; - - - строящиеся «островные» объединения энергосистем Западной Европы; -x-x- возможное объединение энергосистем для обмена приливной энергией; I — приливная энергия, поглощаемая непосредственно энергосистемой; II — сизигийная энергия, аккумулируемая водохранилищами речных ГЭС для длительного регулирования; III — сизигийная приливная энергия, возвращаемая в энергосистему речными ГЭС в квадратурный период; 1 — энергия ПЭС; 2 — энергия ГЭС; 3 — энергия ТЭС и АЭС.

Figure 1-3. Energy exchange scheme between river and tidal electric power plants from the example of the Cotentin tidal power plant [3].

a - 50 million kW; b - possible TPP capacity; c - possible consumption of electric power in Western Europe; d - consumption, million kW; — — — existing "insular" associations of power grids in Western Europe; - - - "insular" power grid associations under construction in Western Europe; -x-x- possible unification of power grids for exchange of tidal power; I - tidal power consumed directly by the power grid; II - syzygial power stored by river hydroelectric plant reservoirs for long-term regulation; III - syzygial tidal power returned to the energy grid by river hydroelectric plants during quadrature; 1 - TPP power; 2 - hydroelectric plant power; 3 - thermal and atomic plant power.

Calculations show [3] that in view of the briefness of the period of intramonthly tidal inequality (only slightly more than 7 days from syzygy to quadrature) and also in view of the relatively small magnitude of the deviation of the mean-syzygy and mean-quadrature amplitude from the mean (if the mean amplitude is designated as 1 then the mean-quadrature magnitude is 0.69 and the mean-syzygy magnitude is 1.23) the required power engineering volume of the water reservoir for this regulation will be only 1% of the annual production of the TPP and may be achieved by more intensive utilization (26 times per year) of the existing seasonal volume without increasing the normal backwater level. The magnitude of the additional hydroelectric power plant capacity needed to utilize the water saved in the river reservoirs of the hydroelectric power plant depends on the state of the actual energy grid and the sharpness of the peak handled by the TPP. The maximum magnitude of this capacity is determined by fluctuation of the tidal level by $1/3$ from mean quadrature to the mean syzygy. Thus, the additional capacity of the river hydroelectric power plant needed to compensate the TPP should be not more than 0.33 of the capacity of the tidal power plant. /16/

It is evident that such unified TPP-HPP operation with exchange of energy throughout a month requires the presence in this system of a hydroelectric power plant with a reservoir suitable for multi-year regulation. This is what determines the necessity and possibility of including a tidal power plant in energy grids which cover large geographical regions, since such grids are the only ones which could have reservoirs capable of multi-year regulation at their disposal.

Inasmuch as France itself does not have such reservoirs and, due to natural conditions, they cannot be created, it seems to us it is namely for this reason that it is impossible to include the Chausey TPP, with its weekly fluctuations in capacity from 3 to 12 million kW, into the French energy grid.

The second reason, the cost of the TPP, was also a very significant obstacle on the way toward realizing the Chausey project. The Rance power plant cost 480 million francs, which comes to 2,000 francs per one installed kW. Although this is two and one half times more than the cost of 1 kW from a river hydroelectric power plant (for example, Gerstheim - 800 francs per kW), it would still be justified in view of the higher value (by 3 times) of the peak capacity produced by the Rance TPP. The whole point is that the quantity of this (peak) energy, fed directly into the grid, is a relatively small portion (20%) of the total energy. Consequently, in order to have a sound economic basis for the construction of a TPP there must be a large decrease in its cost. /17/

This was the problem being solved during the construction of the experimental Kislaya Guba Tidal Power Plant.

1-2. Tidal Energy Resources in the USSR

Determination of the potential of the energy resources of the tide according to L.B. Bernshtein's formula [3, 20] shows that the capacity N and the output E of a tidal power plant for a cadastral evaluation of the site depends on the amplitude A , m , and the potential catchment area of a TPP basin F , km^2 :

$$N, E = kf (A_{mean}^2 F).$$

A survey of tidal phenomena along the seacoasts of the USSR shows that amplitudes useful for power engineering utilization (higher than 4 meters) are observed on the Murmansk Coast (7 meters), in the northeastern part of the White Sea (Gulf of Mezen, 10 meters) and in Shelikhov Bay in the Sea of Okhotsk (Penzhina and Gizhin Bays, 10-11 meters). The height of the tide in these bays is somewhat smaller than the record tides in the Bay of Fundy

(16 meters), the Rance estuary (13.5 meters) and in the Bristol Channel (14 meters), where the most powerful TPPs are being planned but the area of water which can be isolated by TPP dams is so significant that the world's largest TPPs can be built along the coasts of the Okhotsk and White Seas. They can produce 210 billion kW hours per year of the 1,240 billion kW hours per year which can potentially be produced by TPPs along all of the world's tidal seacoasts [20].

The greater part of these resources, 170 billion kW hours per year, is concentrated in the Sea of Okhotsk. Super-powerful TPPs can be built there whose worth will be proven in the more distant future in association with river hydroelectric plants of comparable capacity of the Lena, Kolyma, and Amur Rivers or for the transmission of energy in a program of international cooperation.

Utilization of the energy resources of the White Sea (40 billion kW hours per year), which could be incorporated into the unified energy grid of the European part of the USSR, could be of practicable interest in 20-30 years. The presence of river hydroelectric plants with seasonal regulation makes it possible to use them for compensating the intrasyzygial dips in TPP output, which could be an important component of the system. The absence of damage from flooding during the creation of TPP basins and the capacity to obtain significant quantities of peak energy predetermine the worth of these sites.

/18/

Along the White Sea coast the construction of TPPs is being examined in Lumbovka Bay and the Gulf of Mezen (fig. 1-4).

The Lumbovka TPP can be created by closing off the bay, which is 70 km² in area, with 5.2 kilometer-long dams. The tidal range in this area attains 7.2 meters. Approximately 800 million kW hours can be produced at

this site. From an installed capacity of 320 thousand kW, 280 thousand kW can be produced at the peak of the curve.

There are plans for joint operation of the Lumbovka TPP with hydroelectric power plants on the nearby Ponoï and Iokanga Rivers and also for utilizing the TPP basin to create a second port on the Murmansk coast. However, in view of the fact that construction of the above mentioned river hydroelectric power plants is not envisaged in the near future due to the imminent start-up of the Kola atomic power plant, and also taking into account the relatively small capacity which can be obtained at the Lumbovka TPP, transmission of its peak energy to Leningrad is found to be unwarranted and the question of the Mezen TPP has priority over the Lumbovka installation.

Separating the shoal water of the eastern part of the Gulf of Mezen (amplitude up to 10 meters) by means of a 100 kilometer-long dam (with an average height of 15 meters) can produce a quite significant amount of energy (36 billion kW hours per year).

If a two thousand km² TPP basin is used in accordance with the double-basin scheme the quantity of energy decreases to 20 billion kW hours, but it may be directed to cover the half-peak zone of the load curve with 4-5 thousand hours of utilization if 350 units are installed, each with a capacity of 15 thousand kW.

With possible simplification of these units in comparison with bulb-type units and with mass production their cost will be significantly lowered.

Such an installation, ensuring a solution to the problem of handling the fluctuating part of the load curve for northwestern and central USSR, can be examined in terms of the next 20-30 years. Prior to that time, however, the Mezen-Kuloi estuary TPP may be built. Isolation of the southern estuarine

part of the bay (estuary of the Mezen and Kuloi Rivers) by means of a 25 kilometer-long dam can result in a capacity of 4 million kW and power of up to 10 billion kW hours per year, which is required to handle the peak loads of the northwestern USSR, in joint operation with the Mezen TPP and powerful river hydroelectric and thermal power plants planned for this region.

Understandably, the construction of this installation under the conditions of this unpopulated region, with severe climate and the effect of storm winds and heavy ice, is an extremely complex and expensive problem.

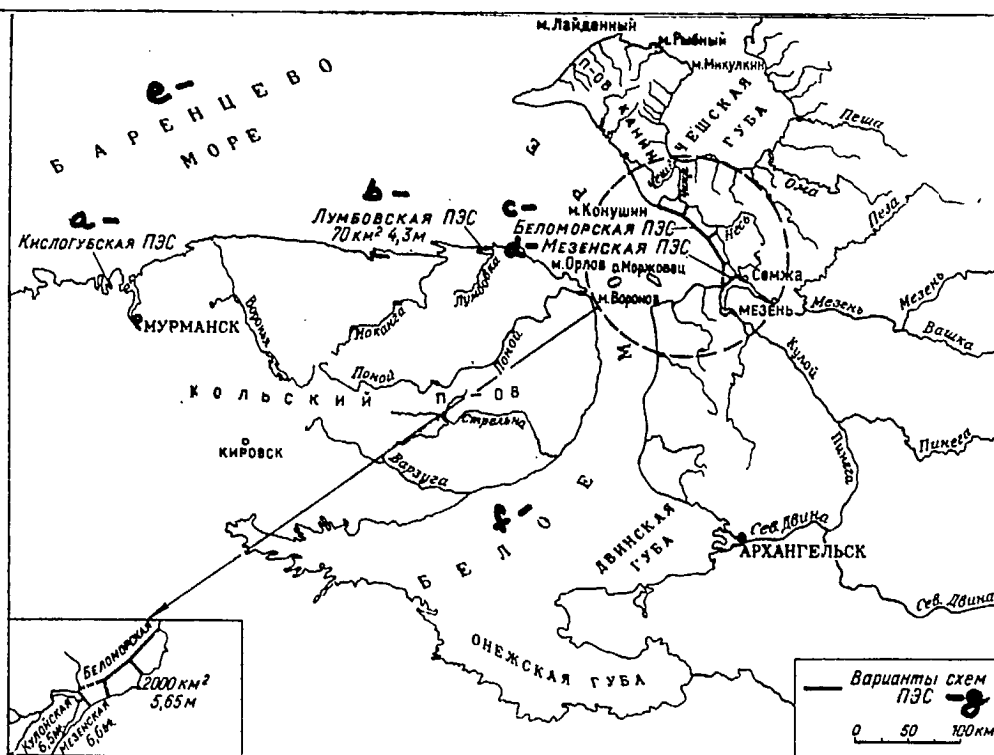


Рис. 1-4. Возможные створы ПЭС на Беломорском побережье [Л. 3].

Figure 1-4. Possible TPP sites on the White Sea Coast [3].

a - Kislaya Guba TPP; b - Lumbovka TPP; c - White Sea TPP; d - Mezen TPP; e - Barents Sea; f - White Sea; g - TPP scheme variants.

For this reason all our research was directed toward finding basic solutions which would permit the construction of a TPP under such conditions in a technically and economically sound manner. This long search led to the conclusion that the most practicable solution to construction under these conditions must be the floating method, which consists in building the TPP structure at an industrial center and delivering it to the site in a ready state with assembled equipment. This method obviates the necessity of constructing huge coffer dams for separating the TPP basins from the sea which, for example, consumed 30% of the total estimated cost of the Rance TPP. In addition, the transfer of the basic work away from the severe conditions at the site will decrease its cost by half.

The floating method, which has previously been successfully employed in marine hydraulic engineering (giant blocks for the construction of breakwaters and coastal protection works), was proposed as early as the 1930's for the erection of individual structural elements of hydroelectric power plants (proposed by L.B. Bernshtein in 1939 for the buttresses of the Kislaya Guba TPP [8]). Engineer V.L. Moshkovich later proposed the construction of the Yaroslavl hydroelectric power plant using giant blocks [34]. B.K. Aleksandrov, corresponding member of the Soviet Academy of Sciences, proposed in 1943 the construction of the Nizhnyaya Kama hydroelectric power plant in the form of a single floating block. Professor K.A. Mikhailov proposed the construction of the Gorki hydroelectric power plant using scow-like blocks which would be manufactured at the prefabricated reinforced concrete plant of the Rybinsk hydroelectric power plant and floated down the Volga. A similar solution was proposed by M.A. Malyshev, Doctor of Technical Sciences, for the Vilyui hydroelectric power plant. It was, however, not possible to realize all of these proposals due to the great weight of the assembled block, necessitated

by the complex configuration of the turbine duct, which is unavoidable when using a vertical unit, or as the result of the large width of the block in the initial variants of horizontal unit construction. It was only in recent years, on the basis of both foreign experience in the construction of marine hydraulic structures using the floating method (the underwater tunnels on the Fraser River in Canada and Rendsburg in West Germany, the dry dock in the port of Genoa in Italy, dams in the Delta-plan in Holland), and Soviet experience (construction of reinforced concrete floating docks), that it has become possible to plan and realize for the Kislaya Guba TPP a new floating design of the structure of the low-head and tidal electric power plant, proposed by L.B. Bernshtein [7]. /21/

The light skeletal design of the reinforced concrete floating dock was offered as a prototype multi-unit TPP block which could be built in a construction dock (for example in the estuary of the Lavna River which flows into Kola Bay opposite Murmansk) and with a draft of 10-14 meters could be delivered by sea to the Mezen estuary site. The immensity of the problem (75 blocks, each 70 x 70 x 30 meters and weighing 70 thousand tons) and the complexity of its solution required the performance not only of theoretical computations and research but also the execution of a full-scale experiment. Despite much experience in the construction of marine structures by the floating method, prior to proceeding with the construction of a powerful industrial TPP by this method an experimental test was required because no one in the world had built an electric power plant in this way. This was the reason for the decision in 1962 to build the Kislaya Guba tidal power plant not far from Murmansk.

This book offers the reader an account of the basic, most important aspects of the project and the experience gained during the construction and testing of the Kislaya Guba tidal power plant which, we profoundly believe, as

as inscribed on the fronton of the TPP building, "Opens the way toward harnessing the powerful energy of the White Sea tides".

Chapter 2

DESCRIPTION OF THE CONSTRUCTION REGION

2-1. Justification of the Kislaya Guba site for an experimental TPP

/22/

Construction of an experimental TPP in Kislaya Guba, proposed as early as 1938 [3,8], was justified by the following considerations.

The capacity and dimensions of an experimental TPP should not be very large but they should be sufficient to serve as a standard for industrial tidal electric power plants.

The experimental plant site should be located as close as possible to an industrial center (Murmansk) and existing transmission lines of the energy grid, while the configuration of the basin and the channel connecting the basin with the sea should permit the experiment to be carried out at relatively minimal cost.

Correlation of several contemplated variants of TPP construction with a schematic electrification plan for fishing and other settlements along the coast revealed the merits of the Kislaya Guba site.

The following sites were examined: Ozerko Bay, Titovskaya Guba, Dal'nezelenitskaya, Yarnyshnaya, Ivanovka (Drozdovka) and Porchnikha. All of these bays have a water surface area not much different from that of Kislaya Guba (1 km^2), somewhat greater tidal ranges (A_{mean} approximately 4 meters), but are connected to the sea by wide channels composed of alluvial deposits. The only exception is Ivanovka Guba, which is connected to the sea by a narrow rocky channel. Its large area (24 km^2) with an average range of 3.4 meters makes it possible to obtain 19 million kW hours at a capacity of 70 thousand kW but, due to its remoteness from electric transmission lines, it is impossible to use this site. This latter circumstance was also the reason for rejecting the other sites.

Kislaya Guba, which is situated near Murmansk and is connected with the sea by a narrow channel, was the most suitable location. The small tidal range (1.1-3.9 meters) made it possible to test the operation of units at minimal heads, which are difficult to utilize. In addition, the narrow channel made it possible to create the TPP basin at relatively minimal cost by cutting it off from the sea by the TPP structure itself.

It should be noted that in other countries also, where tidal power /23/ plants have been and are being planned, construction sites were proposed where this experiment could be carried out at minimal construction cost rather than where the greatest tidal ranges occurred. Thus, the Argentinian commission for the utilization of tidal energy proposed the construction of an experimental TPP at Deseado, where the mean tidal range is 3.54 meters, although the mean tidal range at the nearby port of Gallegos is twice as large. The site proposed for an experimental TPP in England was Chichester, where the amplitude ($A_{\text{mean}} = 3.3$ meters) is also significantly smaller than in the Severn estuary ($A_{\text{mean}} = 8.9$ meters).

A proposal to build an experimental TPP by installing tidal turbines between the spans of the Kola highway bridge was also examined. This would have made it possible to obtain a double effect from the experimental installation: firstly, by utilizing the tidal fluctuations and, secondly, as a result of the decrease in the tidal fluctuations in the tail water of the Tuloma hydroelectric power plant, which would result in decreased head on the turbines. This proposal was also not accepted because its realization would have required relatively greater expenditures for damming the head of Kola Bay. This proposal, however, may be of interest for other low-head hydroelectric power plants with tail water backed up by the tide.

A double tide TPP scheme was also examined as an experimental installation variant¹. In order to carry out this scheme it would have been necessary to excavate a 10 kilometer-long canal through the low-lying necks separating the Rybachii Peninsula (between Vaida and Ozerko Bays) and the Srednii Peninsula (between Kutovaya and M. Volokovaya Bays) from the mainland. However, the insignificant displacement of the tidal phase at these locations does not permit the efficient utilization of this scheme.

Understandably, the creation of an experimental TPP basin in the region of the larger water surfaces of the Lumbovska Bay or the Gulf of Mezen would have required excessively greater expenditures. The link-up of the experimental TPP with the energy grid, if it were located in these bays, would have been totally unrealistic.

The detailed research carried out from 1938 to 1965 on the natural conditions at Kislaya Guba and the project specialists indicated the correctness of the site chosen for the experimental TPP (Fig. 2-1).

There is a hardly noticeable break in the cliffs along the eastern shore of Ura Guba. This "throat" is the entrance to Kislaya Guba (Fig. 2-2). The wide basin (150 meters) immediately becomes a narrow channel (at the low-water line the distance between banks is 35 meters). The length of the channel is 450 meters. At its southern end it widens out again in a funnel-shaped manner to join the large and deep reservoir of the Kislaya Guba, (the water surface area of Kislaya Guba is 1.1 km^2 , the depth reaches 35 meters).

/25/

¹The principle of double tide TPP operation was proposed in the TPP projects of Lesse and San-Jose [3].

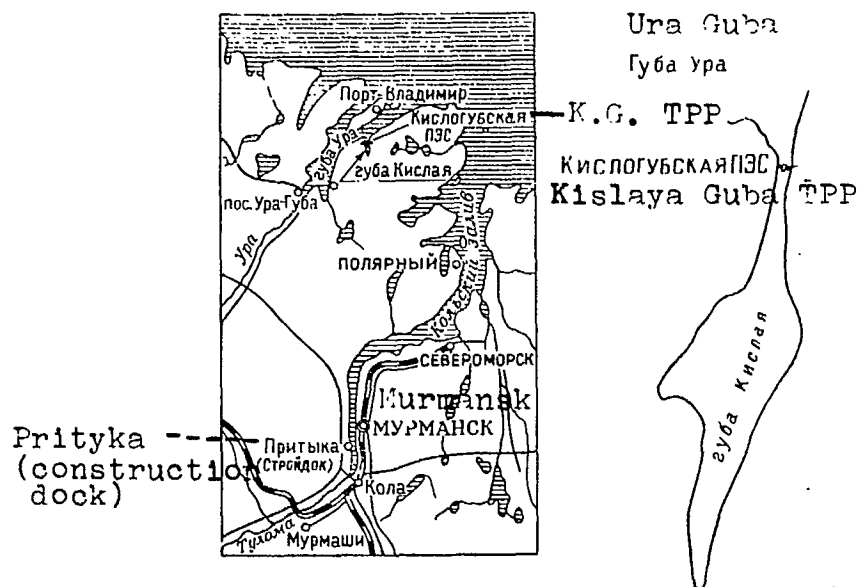


Рис. 2-1. Схема района строительства ПЭС и контуры губы Кислой.

Figure 2-1. Sketch of the TPP construction region and contours of Kislaya Guba.



Рис. 2-2. Панорама губы Кислой до сооружения ПЭС (1960 г.). Фото Л. Б. Бернштейна.

Figure 2-2. Panorama of Kislaya Guba prior to construction of the TPP (1960). Photographed by L.B. Bernshtein.

2-2. Natural Conditions at Kislaya Guba

a) Climatic conditions

The climate of the Kislaya Guba region is marine and polar with a relatively mild winter and a cool overcast and humid summer. The transition of the mean diurnal temperature through 0°C occurs at the end of April and in mid October. The warm and cold periods have an almost equal semi-

annual duration. The winter is characterized by mild frosts and intermittent thaws although wind lulls do occur and the temperature may drop to -30°C and lower while, in the summer, days with temperatures up to $+30^{\circ}\text{C}$ do occasionally take the place of cool overcast weather.

The polar night lasts from the end of November to the middle of January and the polar day lasts from May through July.

Due to the lack of long-term meteorological observations at Kislaya Guba the characteristics of the climate in the construction region over many years (table 2-1) was obtained by various statistical methods from long term observations at the Kola Peninsula Meteorological station.

The regions wind regime is determined by the nature of the atmospheric circulation of the Barents Sea and the Kola Peninsula. During the cold part of the year southerly and southwesterly winds prevail; during the warm period northerly winds prevail. Strong winds (from 15 meters per second and greater) are observed annually from September to May. The mean annual number of days with strong winds is 30. The prevailing direction of storm winds is southerly. The maximum speed of southerly and northerly winds in the region of the TPP may exceed 50 meters per second (frequency 0.1%), as was observed in January of 1963. The speed of wind with a frequency of 2% is 34 meters per second while that of 1% is 40 meters per second.

b) Hydrological conditions

Prior to construction of the TPP the sea water rushed toward the inlet during flood. The channel could not accomodate the flow and, as the level increased, an overfall formed at the narrowest point. At night the swirling flow sparkled with phosphorescent light. The rates of flow at spring tide attained 3-3.5 meters per second while the discharge attained 300 m^3 per second.

Table 2-1.

Climatic description of the Kislaya Guba region.

Климатические характеристики района губы Кислой

a - Показатели	b - Наименование величин	c - Значения по месяцам												g - Среднего- довое
		I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
d - Температура воздуха, °C	h - Средняя	-10,0	-10,3	-7,4	-2,2	2,5	7,7	11,3	1,5	6,0	0,2	-4,5	-5,2	-0,4
	i - Максимальная	8,0	5,0	8,0	13,0	25,0	29,0	30,0	26,0	24,0	12,0	9,0	6,0	30
	j - Минимальная	-33,0	-35,0	-32,0	-24,0	-13,0	-5,0	-2,0	-1,0	-10,0	-22,0	-27,0	-31,0	-35
e - Количество осадков, мм		84	70	25	30	36	49	61	65	65	57	49	36	515
f - Атмосферное давление, мм рт.ст.	h - Среднее	1 030	1 007	1 005	1 036	1 019	1 008	1 008	1 037	1 033	1 003	1 007	1 002	1 005
	i - Максимальное	1 042	1 045	1 033	1 034	1 045	1 031	1 023	1 025	1 027	1 026	1 041	1 042	1 045
	j - Минимальное	951	940	946	971	984	979	983	979	943	942	963	930	954

a - indicators; b - denomination of values; c - values by month; d - air temperature, °C; e - amount of precipitation; f - atmospheric pressure, mm bar; g - mean annual; h - mean; i - maximum; j - minimum.

At high water the flow stopped and, in calm sunny weather, the rocks covering the channel bed could be clearly seen through the mirror-smooth transparent water. Several minutes later the flow, awakened by the ebb, turned in the opposite direction, from the inlet to the sea. At low water the depth in the narrowest and shallowest part of the channel was 2 meters.

Instrument observations of the currents, carried out in 1938, 1939, 1960 and 1964, showed that the ebb-flow sequence of the currents occurred one hour after high and low water, while the maximum speeds were observed 1.5 hours prior to these levels. During the mean amplitudes period the maximum speeds (as measured in December 1938) were 3.08 meters per second.

In August 1960, during the period of syzygy, a speed of 2.68 meters per second was recorded. The discharges of the tidal wave at the TPP site were determined by harmonic analyses of observations of the current made in 1960 and 1964. The mean discharges of water during the flood and ebb cycle were: 148 m^3 per second during syzygy, 69 m^3 per second during quadrature and 116.8 m^3 per second in between (Fig. 2-3).

Synchronous observations of the level have shown that the tide fills the entire water surface area of the inlet without phase displacement in terms of level or time. /28/

Repeated observations of the tides over a four-month period in 1939 and over nine months in 1967-68 (during the operating period observations are carried out constantly with automatic recorders) have shown that the ebb-flow cycle has a regular semi-diurnal character [during a lunar day (24 hours 50 minutes) there are two floods and two ebbs]. The duration of dropping level is equal to the duration of increasing level. The maximum duration of stand at low (or high) water attains 15-20 minutes during small (quadrature) amplitudes (Fig. 2-3).

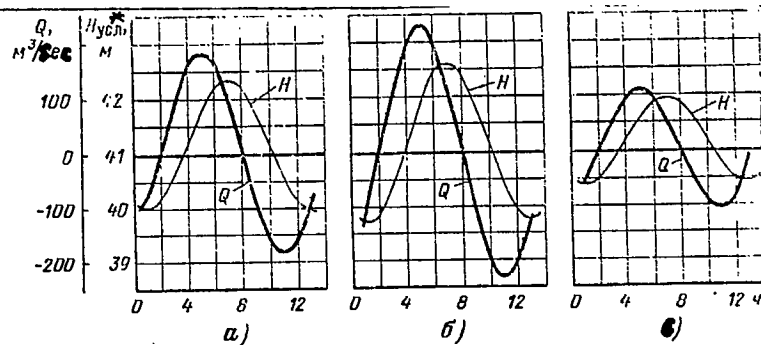


Рис. 2-3. Колебания уровней и расходов воды в створе ПЭС в различных фазах прилива.

а — средняя амплитуда; б — средняя сизигийная амплитуда; в — средняя квадратурная амплитуда.

Figure 2-3. Fluctuations in the levels and discharges of water at the TPP site during different tidal phases.

a - mean amplitude; б - mean amplitude at syzygy; в - mean amplitude at quadrature; * - arbitrary.

The maximum syzygial tide (tide-producing forces of the sun and moon acting in one direction) is 3.96 meters, the mean syzygial is 2.86 meters, the mean is 2.27 meters, and the mean quadrature is 1.58 meters (Fig. 2-3). The minimum quadrature tide (tide-producing forces of the sun and moon acting in opposite directions) is 1.07 meters (Fig. 2-4).

The periodic fluctuations induced by astronomical causes lend themselves to precise calculation and precomputation which is published in special tide tables.

It must be taken into account that fluctuations of level caused by tides are also affected by non-periodic factors (atmospheric pressure, off-and-on water) which can result in deviations of the extreme levels of ± 0.5 meters.

Thus, the actual water level in Kislaya Guba is determined by the sum of the component levels from the action of periodic and non-periodic fluctuations. Inasmuch as the former lend themselves to quite precise computation (harmonic analysis) on the basis of existing observations carried out over many months, while the latter require observations over many years, which are lacking, determination of the composite extreme levels, required for the design of the TPP, was carried out by the method of mathematical statistics. In order to do this a short series of observations at Kislaya Guba and observations over many (37) years at a neighboring point-analogue were used. The theoretical frequency curves thus obtained made it possible to establish the estimated extreme levels (Fig. 2-5).

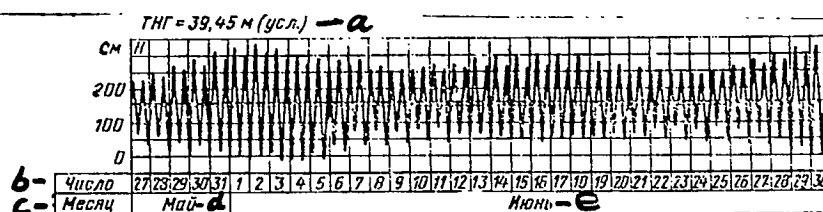


Рис. 2-4. График колебаний уровней воды в створе ПЭС.

Figure 2-4. Graph of water level fluctuations at the TPP site.

a - *theoretical null* (arbitrary); b - date; c - month; d - May; e - June.
depth

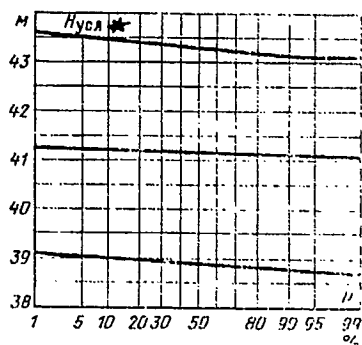


Рис. 2-5. Кривые обеспеченности максимальных, средних и минимальных уровней в створе ПЭС.

Figure 2-5. Frequency curves of maximum, mean and minimum levels at the TPP site. * - arbitrary.

The maximum annual level at a frequency of 5% is 43.50 meters (arbitrary), the minimum is 39.03 meters (arbitrary). The mean level over many years at Kislaya Guba is 41.17 meters (arbitrary). The theoretical null depth¹ is assumed in the design to be 39.45 meters.

Guided by this system of levels, the areas of the water surface were determined between the low and high water levels, and varied from 0.97 to 1.5 km². When the basin is pumped out to the 38.4 meter mark this area decreases to 0.91 km² and when it is pumped up to the 44 meter mark the area increases to 1.7 km² (Fig. 2-6). The gradually and insignificantly increasing water surface area changes sharply after the 42.92 meter mark due to the annexation of Lake Presnoye, which has a water surface area of 0.15 km², and at the 43.32 meter mark by the annexation of Lake Glubokoye, which has an area of 0.63 km².

Due to the orientation of Kislaya Guba along the meridian, its entrance from the north sheltered by the high cliffs of Shalim Island, and the orientation of Ura Guba itself from the southwest to the northeast, even strong storm winds from the west and east cannot give rise to significant

¹The theoretical null depth (TND) is the name given to the lowest possible water level in a tidal sea due to astronomical causes. In this case this level is 39.45 meters in the arbitrary system of heights under which the Kislaya Guba TPP project was built.

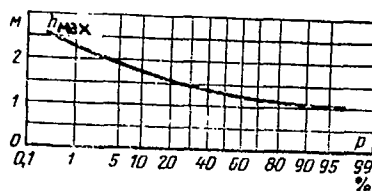


Рис. 2-7. Кривые обеспеченности максимальной высоты волны при ветре северных направлений в створе ПЭС.

Figure 2-7. Maximum wave height frequency curves for northerly winds at the TPP site.

c) Ice conditions

Investigation of the ice conditions is important for determining the effect of ice on the TPP structure and their effect on TPP operation: disruption of the turbine unit regime, the formation of ice crusts, freezing of gates and formation of fast ice, resulting in a decrease of the tidal range and the useful head. Study of the ice regime of the Kislaya Guba region was carried out not only under natural conditions but also under conditions of the inlet being cut off from the sea, which was to occur after the construction of the TPP. Beginning in 1938 the ice regime of Kislaya Guba was studied by Lengidroproekt, the State Oceanographic Institute, the Murmansk Administration of the Hydrometeorological Service and the Winter Regime Laboratory of the All-Union Scientific Research Institute of Hydrology. This work made it possible to establish that, as a consequence of the warming effect of the Gulf Stream, the area of the sea into which Kislaya Guba discharges becomes covered with ice only in its southern part (effect of fresh water). In mild winters the ice was 5-10 cm thick and appeared at the beginning of November, the ice was repeatedly carried away and reformed. During severe winters the thickness of the ice attained 40 cm and the ice reached as far as Shalim Island.

During much of the winter the Kislaya Guba channel and the sea around it were free of ice or were covered by initial forms of ice such as snow slush, ice slush, frazil ice and ice rind, which were constantly carried out to sea by the action of the ebb tides and the wind. Large ice reefs, whose boundary was at a distance of 500 kilometers, were not observed to approach Kislaya Guba (but even if this were to occur they would not be able to affect the Kislaya Guba TPP structure due to the presence of the shoal barrier and the narrowing banks). Under natural conditions fast ice 60-70 cm thick formed annually in the Kislaya Guba basin from November through May. It rose and fell with the tide like a float without diminishing its range.

Naturally, after isolation of the Kislaya Guba by the TPP structures, two characteristic ice condition regimes would be observed: one arising during a lengthy shutdown of the TPP, when there would be no currents drawing ice into the Kislaya Guba channel while freshening would be taking place within the inlet itself, and the regime occurring during the operation of the TPP when the exchange of water in the Kislaya Guba basin would correspond to non-regulated flow.

Study of the first regime was carried out after the Kislaya Guba basin was cut off by a rock-fill embankment created to prepare the underwater foundation (from 1 August 1964 to 21 December 1968). During the severe winter of 1965-1966 (the mean monthly air temperature dropped 9°C below the mean temperature for many preceding years) Ura Guba had a fast ice cover up to 60 cm thick from February on. Shore ice 10-12 cm thick was observed in the Kislaya Guba channel. On some days in February, March and the beginning of April very light floes from Ura Guba packed into the Kislaya Guba channel and, freezing together, formed hummocked fast ice

35-40 cm thick. In the middle of April drifting triangular, rectangular and polygonal ice cakes up to 40 meters square and 55-60 cm thick were observed to the north of Mogil'nyi Island. A continuous sheet of ice 60-90 cm thick and up to 110 cm at the apex formed in the Kislaya Guba basin during that winter but a fast ice foot did not form. By the end of April the ice cleared from the Kislaya Guba channel and the adjacent sea.

Analysis of these conditions permitted the assumption that after construction of the TPP, with continuing exchange of water between the basin and the sea, and insignificant freshwater discharge (mean annual discharge 0.5 m^3 per second) an ice field would be expected to form in Kislaya Guba during the winter with a large lane in the ice from Tyulenii Island to the TPP site, which would rule out the constant action of ice on the structure on the basin side. The structure also cannot be subjected to the action of ice floes of significant dimensions from the sea side. But even an exceptional case of such floes appearing should not be considered in coincidence with established TPP regimes (all gates opened in the direction of the basin) carried out for experimental reasons. On the contrary, it is precisely under such conditions that it would be possible to achieve artificial repulsion of the ice from the site out to sea. These circumstances made it possible to reject ice booms on both sides of the structure.

d) Engineering and geological conditions

The geological structure of the Kislaya Guba TPP construction region is characterized by the development of Upper Archean granites covered by a thin mantle of loose Quaternary deposits: granites-plagio-microclines, medium-grained, partially gneissized, in places run through with pegmatitic veins and broken diabase dykes. Two tectonic fracture zones have been traced in the granites of this region.

The first tectonic fracture zone cuts across the granite massif on the western shore of the Kislaya Guba channel 70 meters to the north of the site. In relief, this zone is manifested by a deep ravine with almost vertical granite walls running in the northwesterly direction. The ravine is composed of loamy material with fine granite gruss. The second tectonic zone is located within Kislaya Guba under a 10-meter-thick layer of Quaternary deposits. This zone is composed of cataclastic granite, crushed down in places to a sandy-argillaceous mass. /33/

The Quaternary deposits are represented by glacial, marine, talus, and bog formations. The glacial deposits have a restricted distribution on the eastern shore of Kislaya Guba, in the area for auxiliary and subsidiary structures. The moraine is composed of rubbly and pebbly material with a powdered sandy and loamy filler. The moraine is 1.2-2 meters thick.

The marine deposits are widely distributed. On the shores their thickness does not exceed 5 meters. In Kislaya Guba the marine deposits lie on the surface of the rocks in a 5-15 meter layer. They are absent only near the eastern shore of Kislaya Guba where the bottom is composed of granites.

The seismic and drilling surveys which were carried out, as well as underwater exploration, made it possible to establish that the Kislaya Guba channel is a ravine, composed of large boulder deposits with a sandy-gravel and partially shelly filler, with its greatest thickness of up to 7 meters along the channel axis, and thinning out to the cliff at the banks (Fig. 2-8).

e) Flora and fauna

The physicogeographical and hydrological conditions, as well as the hydrochemical regime of Kislaya Guba are, on the whole, favorable to the development of littoral and sublittoral vegetation. Strong tidal currents in the channel, however, have an adverse affect on its spread.

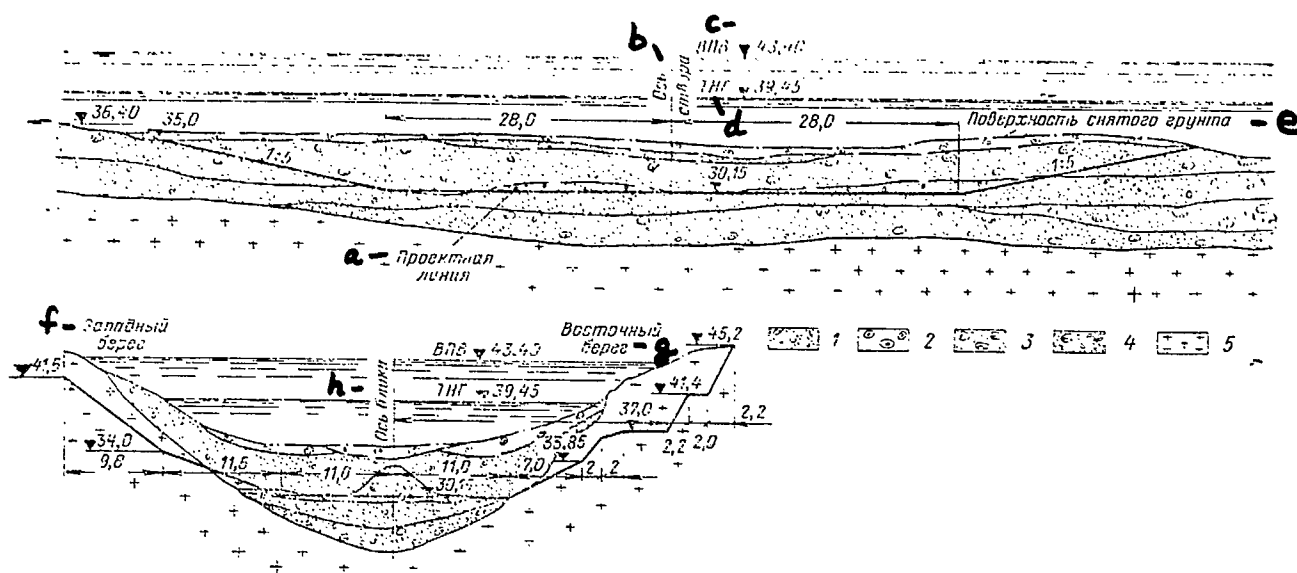


Рис. 2-8. Продольный и поперечный геологические разрезы.

1 — песок светло-серый разнозернистый, мелкозернистый, местами слабослоистый с гравием, галькой и редкими валунами, с раковинами преимущественно раздробленными; 2 — ракушечник; 3 — валуно-галечно-гравийные отложения, заполненные разнозернистым и мелкозернистым песком, с раковинами, преимущественно раздробленными; 4 — песок разнозернистый с гравием, галькой и валунами; 5 — гранит плагно-микроклиновый среднезернистый, крепкий трещиноватый, частично огнейсованный, местами с пегматитовыми жилами.

Figure 2-8. Longitudinal and transverse geological sections.

1 - light grey sand, inequigranular, fine-grained, slightly layered in places with gravel, rubble and rare boulders, with shells, mostly crushed; 2 - coquina; 3 - boulder-rubble-gravel deposits, filled with inequigranular and fine-grained sand, with shells, mainly crushed; 4 - inequigranular sand with gravel, rubble and boulders; 5 - plagio-microcline granite, medium grained, strong, fractured, partially gneissized, in places with pegmatitic veins; a - projected line; b - site axis; c - higher high water; d - theoretic null depth; e - surface of excavated bottom; f - west bank; g - east bank; h - block axis.

Detailed studies have shown that the most important species of littoral seaweeds are fucoides, representatives of the family Fucaceae¹.

There are approximately 160 tons of them in the wet state in Kislaya Guba.

/35/

The annual weight of fucoides torn off in the inlet is approximately 40 tons.

Part of this quantity comes to rest on the shores of Tyulenii Island but

¹The studies were carried out by the Northern Division of the Polar Scientific Research Institute of Sea Fisheries and Oceanography im. N.M. Knipovich (SevPINRO) under the direction of K.P. Gemp, Candidate of Biological Sciences, and the Institute of Oceanology im. P.P. Shirshov AN USSR (IOAN).

Later however, when the Ministry of Energy of the USSR decided to turn over construction of the Kislaya Guba TPP to the Northern Administration for the Construction of Hydroelectric Structures, (Sergidrostroi), it became necessary to find a site for the construction dock along the coast of Kola Bay closer to the Sevgidrostroi base, which was engaged in the construction of two hydroelectric power plants near Murmansk. Several sites were examined at this time which were immediately adjacent to Kombinats manufacturing pre-fabricated houses in Murmansk and to the north of Murmansk. The limited area at these sites forced the search to be concentrated to the south of Murmansk because the shoreline to the north is being or will be used for harbor wharves. /36/ This restricted the choice to a section of coast from the settlement of Drovyanoe to Cape Prityka on the western shore of Kola Bay because the approach to other sections of the shoreline was insufficiently deep. In this section the most suitable site in terms of all the conditions listed above was found to be Cape Prityka. At this site there was a gently sloping platform, 400 meters long and approximately 120 meters wide, squeezed in between the water line of the bay and a good highway connecting Murmansk and Pechenga and situated close to the Sevgidrostroi construction base at Prichal'noe station. The most important reason, however, for the choice of this site to build the construction dock was the depth of 5-6 meters (from theoretical null depth), which was sufficient to float out the blocks on pontoons without dredging.

Climatic conditions. The studies which were carried out showed that the climate at this site was almost identical with that at Murmansk: mild, marine, with a long and relatively warm winter. There occur short significant cold spells to -40°C and below and thaws when the temperature rises to $6-8^{\circ}\text{C}$. During the prevailing cool weather during the summer with temperatures of $0-4^{\circ}\text{C}$, hot days may occur with temperatures up to 30°C and higher. Spring frosts are over, on the average, in early June while fall frosts begin in the first half of October.

The duration of the frost-free period is approximately 100 days. During the cold part of the year southwesterly and southerly winds prevail, while during the warm period northerly winds are predominant. Strong winds may occur at any time of the year but most often during the cold period. On the average there are six days with strong winds per year, in some years there are 5-6 such days per month and up to 18 such days during the year. The estimated annual maximum wind speed with a frequency of 2%, without consideration of its direction, is 24 meters per second.

Fogs are observed during every month of the year but they are most frequent during the cold months. Blizzards begin in October, or sometimes in September, and end in May.

Engineering and geological conditions. The dock construction site is steeply inclined toward the bay and is formed by a moraine lying on a rocky surface. Within the confines of the dock perimeter the moraine is more than 19.5 meters thick. At the shore and underwater parts of the site, in the area of the cut, the surface of the moraine passes beneath a layer of marine deposits of various thickness and heterogenous lithological composition: sands, gravel-pebble-boulder formations, sandy loam.

/37/

In order to have minimum excavation (40 thousand m^3) the foundation area of the construction dock was planned to be situated right at the water line. However, the presence of a small layer of rubble perpendicularly adjacent to the shore caused the general contractor to fear that there would be seepage of water from the bay into the dock along the contact between the embankment and this rubble, and the dock was moved further inland, which resulted in an unfounded significant increase (up to 150 thousand m^3) in the volume of excavation of the foundation area and the gap.

In order to keep the tide from reaching the foundation area it was separated from the bay with the excavated earth. On the moraine and morainic deposits within the confines of the cut, fill is also observed consisting of sand and boulders 0.4 meters in size, but accumulations containing 60% boulders 1-2 meters in diameter also occur.

The dock foundation was laid in a moraine formation with a naturally dense structure, low permeability (permeability coefficient 0.02-3.78, average of 0.58 meters per day) and heterogeneous granulometric composition. Crystalline rock boulders of various dimensions comprise 20-30% of the moraine. Individual boulders are as large as 2-3 meters. In terms of difficulty of working, the moraine belongs to the IVth and Vth category of soils. The stability of the foundation walls is ensured when they are laid at a slope of 1:2.5.

Hydrological Conditions. The water level regime of the southern bend of Kola Bay at Cape Prityka is characterized by periodic fluctuations caused by tides and non-period fluctuations due to the coastal discharge of the Kola and Tuloma Rivers, atmospheric pressure, and other hydrometeorological factors. The periodic components of the cumulative level are determined from a 2-month-long series of level observations at Cape Prityka in 1968.

The tides at Cape Prityka have a regular semi-diurnal character. The period of rising water level is practically equal to the period of falling level. The mean amplitude of the tide at syzygy is 3.21, at quadrature it is 1.72 and the average is 2.53 meters.

The maximum monthly level with a frequency of 1% relative to the theoretical null depth is equal to: 4.27 meters in June, 4.36 meters in July, 4.39 meters in August; the minimum monthly levels are, correspondingly, 0.55, 0.59 and 0.42 meters. The mean level over many years in the southern arm of Kola Bay at Cape Prityka is 2.17 meters above the theoretical null depth.

In order to solve this problem of towing away the floating TPP structures from the dock the characteristics of the tide at Cape Prityka turned out to be very important. The main role in the formation of the cumulative currents in the southern arm at Cape Prityka are played by the incoming and outgoing currents as well as by the constant discharge and drift currents. The direction of the constant discharge currents from Tuloma and Kola Rivers is from the head of the Bay to the estuary and are manifested only in the 0-5 meters layer. During the period of freshets they attain a speed of 1 knot. Drift currents develop during southerly and northerly winds. At wind speeds of 15-20 meters per second the speeds of the drift currents attain 1-1.5 knots. Tidal currents arise almost simultaneously throughout the southern arm of Kola Bay, have a general direction toward its head and last approximately four hours. Within one hour after high water the tide changes from flood to ebb, which lasts approximately 6 hours. The speed of the ebb tide is faster than that of the flood tide. The maximum speed of the flood tide is observed two hours before high water, the maximum speed of the ebb tide is observed two hours before low water. The minimum speeds of the flood and ebb tides occur 1-1.5 hours after high and low water. During syzygy the speed of the cumulative currents at Cape Prityka are 3-5 knots in the upper layer, 2.5-3.5 knots in the 0-4 meter layer, and 2.0-2.5 knots in the 4-5 meter layer.

Chapter 3

MAIN STRUCTURES OF THE KISLAYA GUBA TPP

3-1. Layout of the hydraulic power system and the TPP structure

The favorable natural relief and configuration of the banks forming the narrow channel, with steeply dipping rock on the western shore and a cliff, changing to a small plateau formed by the retreating rocks on the eastern bank, made possible a very convenient layout of the hydraulic power system (Fig. 3-1). The channel is cut off by the TPP structure and adjoining dams which are from 0 to 15 meters high and 35 meters long. The natural basin in front of the entrance to Kislaya Guba forms a convenient approach area which makes it possible to establish a berth for ships on the wide sea side (24 x 20 meters), which provides moorage for ships with a draft of up to 3.4 meters (anchorage for ships and other means of floating transport of any draft and tonnage is accomplished at a roadstead consisting of a buoy with an anchor group). A quite large area, located on the elevated eastern bank and with a smooth approach to it, made it possible to locate there (along the road from the moorage): a fuel and lubricating materials storehouse and a house for operating and research staff, which is designed¹ in such a way that the workers and specialists living in it would not feel cut off from the "wide world". There are sixteen apartments in the house with hot water, heating, a lounge with a flower garden and a fireplace, a movie theatre, television, and automatic telephone exchange. Not far from the house are a garage, storehouses, a standby diesel electric power station, an open switchboard, and a hydrometeorological station. A 600-meter-long water pipeline stretches further into the mountains to a pumping station at the lake and to pressure reservoirs located 70 meters higher than the foundation of the TPP. All of these structures are interconnected by good roads

¹Authors of the project - architects V.N. Merkulov and N.I. Shishkin.

which are paved with asphalt from the docking area to the outdoor electrical distribution system. Flowers and trees were planted at the site from the shore to the living quarters. The bilge pads on which the TPP blocks were towed have been installed as benches on the boulevard along the shore.

Another aim during the design of the Kislaya Guba TPP structure was to create a standard design which would ensure the feasibility of building a tidal power plant by the floating method¹. In order to achieve this the structures had to have the minimum weight and maximum strength necessary during the crossing and at the site in order to bear the design loads. The significance of this project was also taken into account during the construction of the TPP structure as a prototype for low-pressure river hydroelectric power plants with bulb-type units, proposals for the construction of which were put forward in the USSR on the basis of the Kislaya Guba TPP project [5]. Maximum utilization of reinforced concrete in the structure for the throughput of operating and excess discharge, which is also determined by the specific features of the bulb-type unit, acquired special significance. /41/

The requirements listed above were the basis for the throughway design of the TPP structure [7]. In developing this design most attention was devoted to compact disposition of the equipment and minimal constriction of the pressure front by the bearing elements of the structure. This condition is fully satisfied by the need to ensuring minimum weight of the structure and its structural strength, achieved not as a result of its build but rather with the aid of the spatial performance of its thin-walled elements.

1

The Kislaya Guba TPP structure was designed by a collective of the Department of Tidal Power Plants of the All-Union Planning, Surveying and Scientific Research Institute including engineers E.D. Zhukov, Yu.F. Sychugov, A.M. Pirogov, N.P. Solomatin, N.A. Kirillov, Candidate of Geographical Sciences V.E. Prival'skii, Candidate of Technical Sciences L.I. Suponitskii and others under the direction of Chief Project Engineer L.B. Bernshtein.

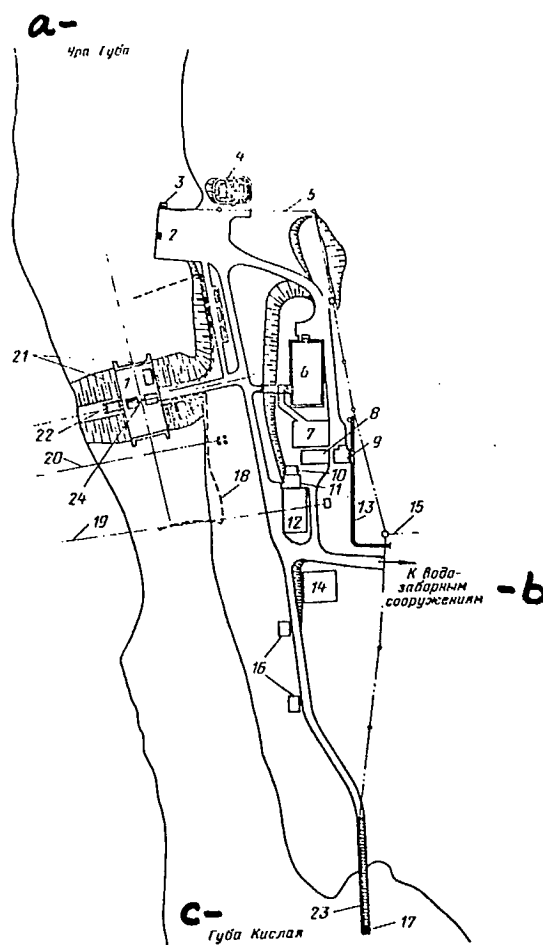


Рис. 3-1. Схема расположения гидроузла.

1 — здание ТЭС; 2 — причал; 3 — мареограф в бассейне; 4 — склад горюче-смазочных материалов; 5 — линии энергоснабжения и связи к мареографу в бассейне; 6 — дом эксплуатационного и исследовательского персонала; 7 — спортивная площадка; 8 — детская площадка; 9 — овощехранилище; 10 — дизельная резервная электростанция; 11 — щитовая; 12 — подстанция ОРУ 35 кВ; 13 — внешние линии водовода; 14 — гидрометеостанция; 15 — линии энергоснабжения и связи к мареографу в вершине губы Кислой; 16 — складские помещения; 17 — мареограф во внутреннем бассейне; 18 — анод катодной защиты; 19 — линии электропередачи 35 кВ; 20 — линия связи; 21 — сопригающие дамбы; 22 — затворохранилище; 23 — причал для внутреннего рейда; 24 — станция катодной защиты.

Figure 3-1. Layout of the hydro engineering complex.

1 - TPP structure; 2 - dock; 3 - tide gauge in basin; 4 - fuel and lubricating materials storehouse; 5 - electric power lines and basin tide gauge lines; 6 - house for operating and research staff; 7 - athletic field; 8 - children's playground; 9 - vegetable depository; 10 - standby diesel electric power station; 11 - switchboard; 12 - outdoor electric distribution sub-station, 35 kW; 13 - above-ground water supply lines; 14 - hydrometeorological station; 15 - electric power lines and lines to the tide gauge at the head of Kislaya Guba; 16 - storehouses; 17 - tide gauge in the inner basin; 18 - anode of cathodic protection; 19 - 35 kW electric power line; 20 - communications line; 21 - adjoining dams; 22 - gate chambers; 23 - inner roadstead dock; 24 - cathodic protection station; a - Ura Guba; b - to water intake station; c - Kislaya Guba.

Such stability of the TPP structure can only be achieved by loading the lightened structure with ballast (soil).

These problems are solved by the layout of the Kislaya Guba TPP structure shown in the zone of unit No. 1 (Fig. 3-2, a), above which there is a surface spillway with pressurized-oil and control systems beneath its floor slab.

It should be mentioned that the advent of this layout had a determining effect on the evolution of low-head river hydroelectric plant buildings which incorporate spillways. Thus, in previously built integrated hydroelectric power plants the level of the spillway slab was higher than the level of the tail race by 20 meters (Pavlovskaya HPP), 24 meters (Iriklinskaya HPP), 11 meters (Kamskaya HPP) and 10 meters (Cherepovetskaya HPP). But in the latest (completed) Kiev HPP project, as a result of the influence of the Kislaya Guba TPP project in which the spillway level is 1.8 meters above the low-water mark, this distance was decreased to 4.5 meters.

The floating block of the two-unit structure of the Kislaya Guba TPP is 36 x 18.3 meters in plan and 15.35 meters high. Figures 3-2 to 3-4 show cross-sectional and plan views of the TPP structure from which the block layout may be seen. It contains the hydroelectric power and mechanical parts, the surface spillway and the bottom weir, the control apparatus for the mechanical and electrical equipment of the TPP, the TPP control panel, the pressurized oil system, the terminals of the cables from the control gauge apparatus to the control and automation panel, the ventilating, draining and water systems, the cathode protection apparatus and lines, and the gate-slot oil heating system. /48/

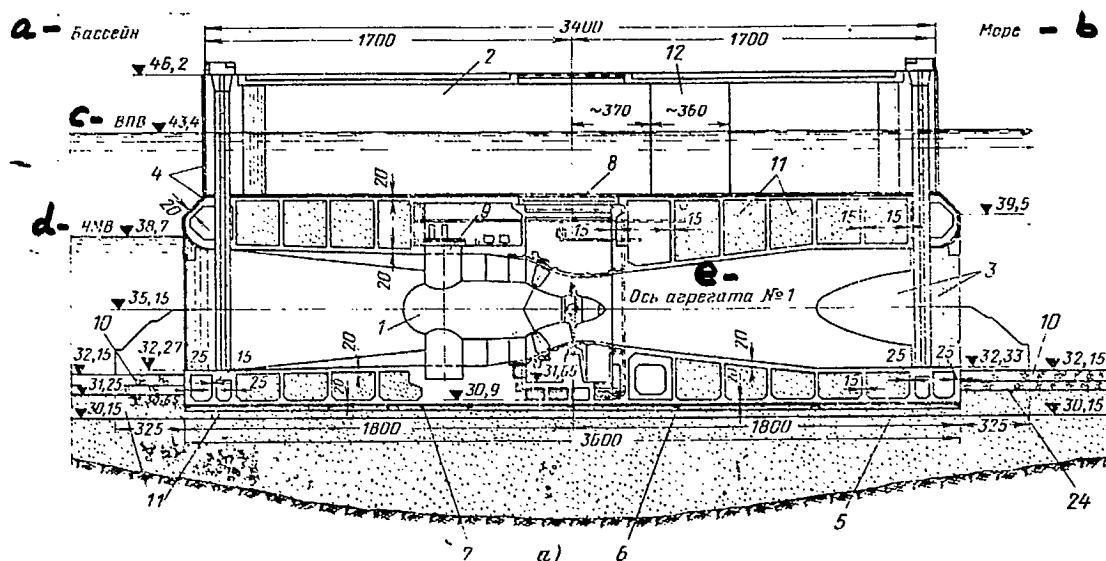


Рис. 3-2. Продольный разрез по зданию ПЭС.

а — по оси агрегата № 1; б — по оси агрегата № 2; 1 — обратимый капсульный агрегат $N=400$ квт, $n=72$ об/мин, $D_1=3.3$ м; 2 — водослив; 3 — гидроизоляция; 4 — теплоизоляция; 5 — ацетатные плиты; 6 — противосифонная уголково-рама; 7 — система цементации; 8 — герметическая крышка; 9 — шахта доступа; 10 — крепление карьерным камнем; 11 — песчано-гравийная смесь; 12 — опытный участок для испытания теплогидроизоляционных покрытий; 13 — портално-стреловой кран грузоподъемностью 30/5 тс; 14 — пульт управления катодной защитой и контрольно-измерительной аппаратурой ПЭС; 15 — пульт

Figure 3-2. Longitudinal cross section of the TPP structure.

a — along the axis of turbine unit No. 1.; b — along the axis of turbine unit No. 2;

1 — reversible bulb-type unit $N = 400$ kW, $n = 72$ revolutions per minute, $D_1 = 3.3$ meters; 2 — spillway; 3 — water proofing; 4 — thermal insulation; 5 — acetate pads; 6 — anti-scour angular frame; 7 — cementation system; 8 — hermetic cover; 9 — access shaft; 10 — quarried stone support; 11 — sand-gravel mixture; 12 — open area for testing hydrothermal insulation coatings; 13 — derrick-gantry crane with a lifting power of 30/5 ton-force; 14 — control panel for the cathodic protection and control and gauge apparatus of the TPP; 15 — control panel; 16 — hydraulic drive system; 17 — conical metal conduit prior to installation of unit No. 2; 18 — oil pump assembly; 19 — oil system; 20 — ventilating system; 21 — conduit drainage system; 22 — pumping-out and drainage system; 23 — dense moraine deposits; 24 — transitional layer.

a — basin; b — sea; c — higher high water; d — lower low water; e — axis of unit No. 1; f — axis of unit No. 2.

The space above turbine unit No. 2 (Fig. 3-2, b) was used for locating the electrical equipment and the control panels, inasmuch as it was possible to use only one turbine unit bay for experimental testing of the layout under consideration. (In standard use the control panel and electrical equipment are located in the gallery over the spillway apertures

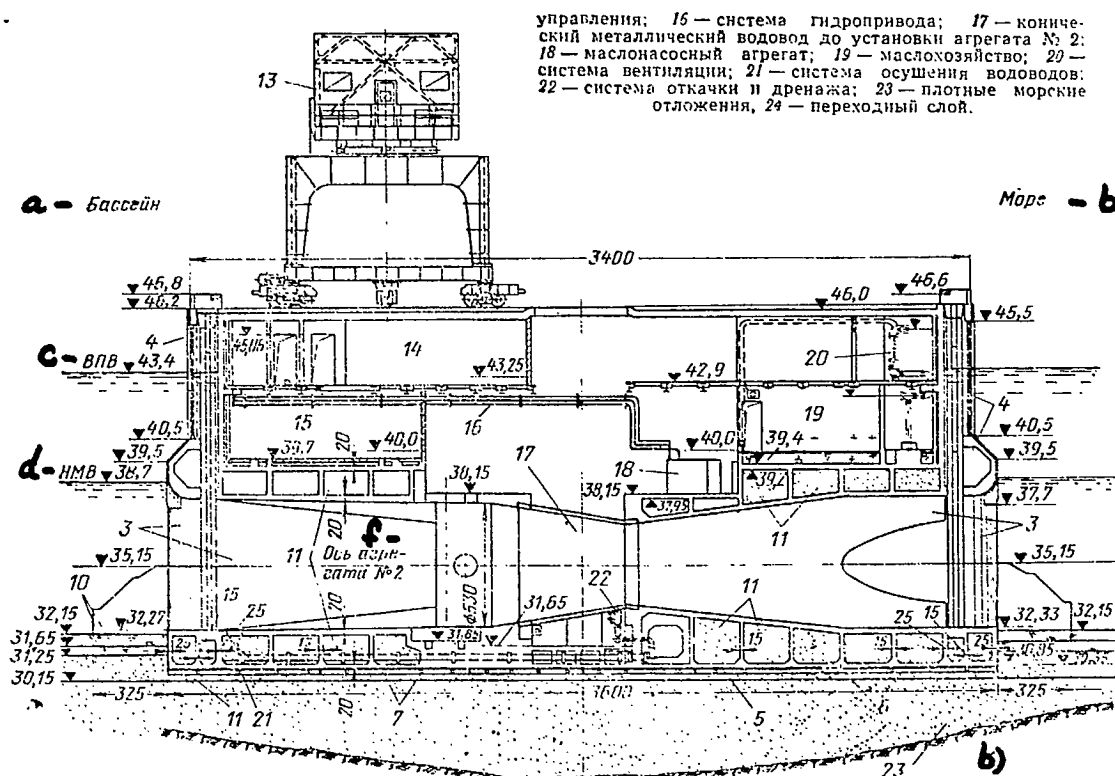


Figure 3-2, b

above the buttresses, as was efficiently accomplished at the Kama hydroelectric power plant).

The layout of the hydraulic power equipment under the conditions of the thin-walled and very confined design of the floating block presented certain difficulties. The reversible bulb-type turbine unit was installed in the reinforced concrete water passage whose configuration was specified by the turbine manufacturer. The rectiaxial conical water conduit, which changes to a rectilinear cross section at the entrance and exit, provides for the input and discharge of water to the runner wheel of the turbine. The entrance caps of the water conduits are elliptical in order to ensure smoother entrance and exit of the flow. The gate slots are located between the elliptical caps of the entrance section and the water conduits. The length of the water conduit was determined with regard to its operation as a suction pipe and was taken to be equal to $5.1 D_1$ (Fig. 3-2, a).

The turbine stator, support column and streamlined access shaft, through which the stress on the turbine is transferred to the structure, are assembled to form a monolithic group. The control system equipment, part of which was specially designed for very constricted conditions, is located in the control chamber between the spillway and the water conduit of the turbine unit at the 38.15 meter mark.

The turbine unit block is in the form of a thin-walled reinforced concrete box of the dock type. The structure consists of a bottom slab 18.3×36 meters in plan, 20 centimeters thick, and located at the 30.65 meter mark (Fig. 3-2, a and b), on which there are 15 centimeter-thick bulkheads parallel to its face. The bulkheads are spaced 1.5 and 2 meters apart (Fig. 3-4, a). Two side walls, also 15 centimeters thick, are located along the long side of the slab for the entire height of the block to the 46.0 meter mark. In the middle part of the slab the side walls are connected from top

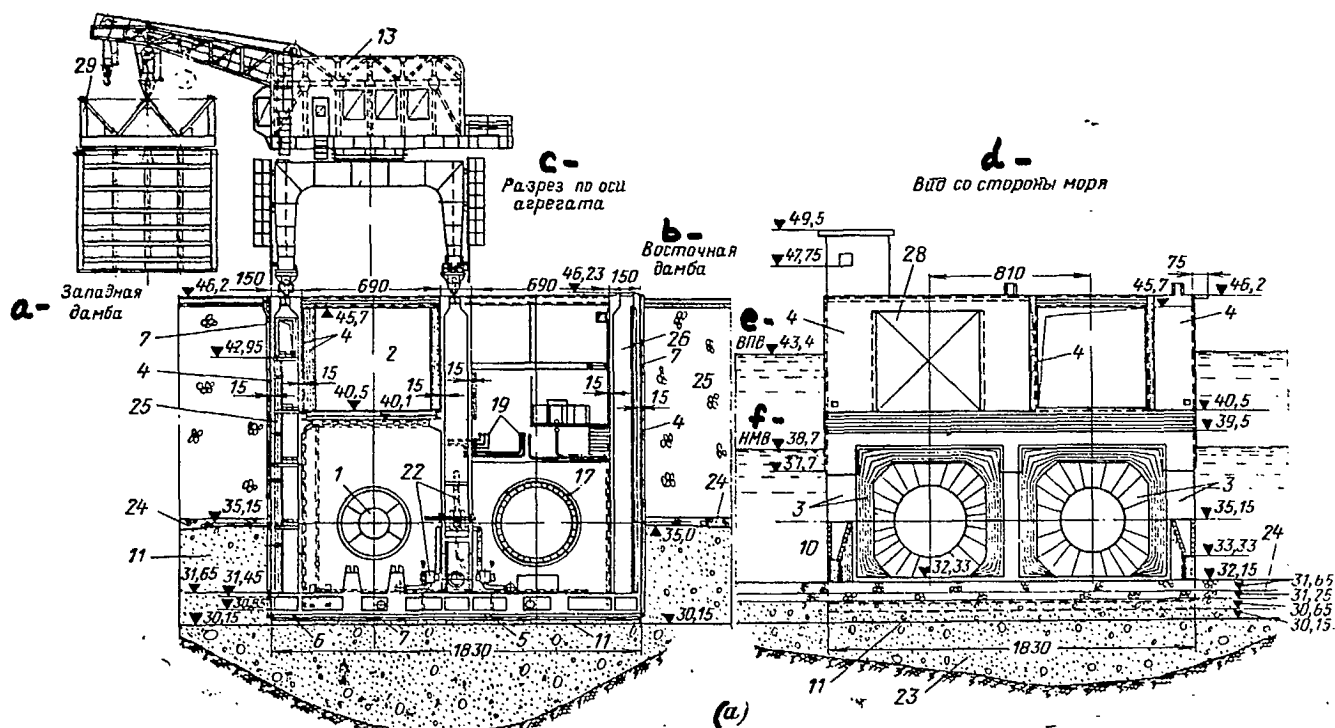


Figure 3-3. Transverse cross sections (a) and plan through the axis of the turbine units (b).

1 - reversible bulb-type unit $N = 400$ kW, $n = 72$ revolutions per minute, $D_1 = 3.3$ meters; 2 - spillway; 3 - waterproofing; 4 - thermal insulation; 5 - acetate pads; 6 - anti-scour angular frame; 7 - cementation system; 8 - hermetic cover; 9 - access shaft; 10 - quarried stone support; 11 - sand-gravel mixture; 12 - open area for testing hydrothermal insulation coatings; 13 - derrick-gantry crane with a lifting power of 30/5 ton-force; 14 - control panel for the cathodic protection and control and gauge apparatus of the TPP; 15 - control panel; 16 - hydraulic drive system; 17 - conical metal conduit prior to installation of unit No. 2; 18 - oil pump assembly; 19 - oil system; 20 - ventilating system; 21 - conduit drainage system; 22 - pumping-out and drainage system; 23 - dense moraine deposits; 24 - transitional layer; 25 - quarried rock fill; 26 - loading shaft; 27 - grappling beam with a load carrying capacity of 30 ton-force; 28 - wooden thermal shield; 29 - sand-gravel mixture fill to the 40.5 meter mark, with hydrophobic soil above; a - west dam; b - east dam; c - cross section through turbine axis; d - view from sea side; e - higher high water; f - lower low water; g - axis of turbine units; h - basin; i - axis of turbine unit No. 1; j - axis of turbine unit No. 2; k - sea.

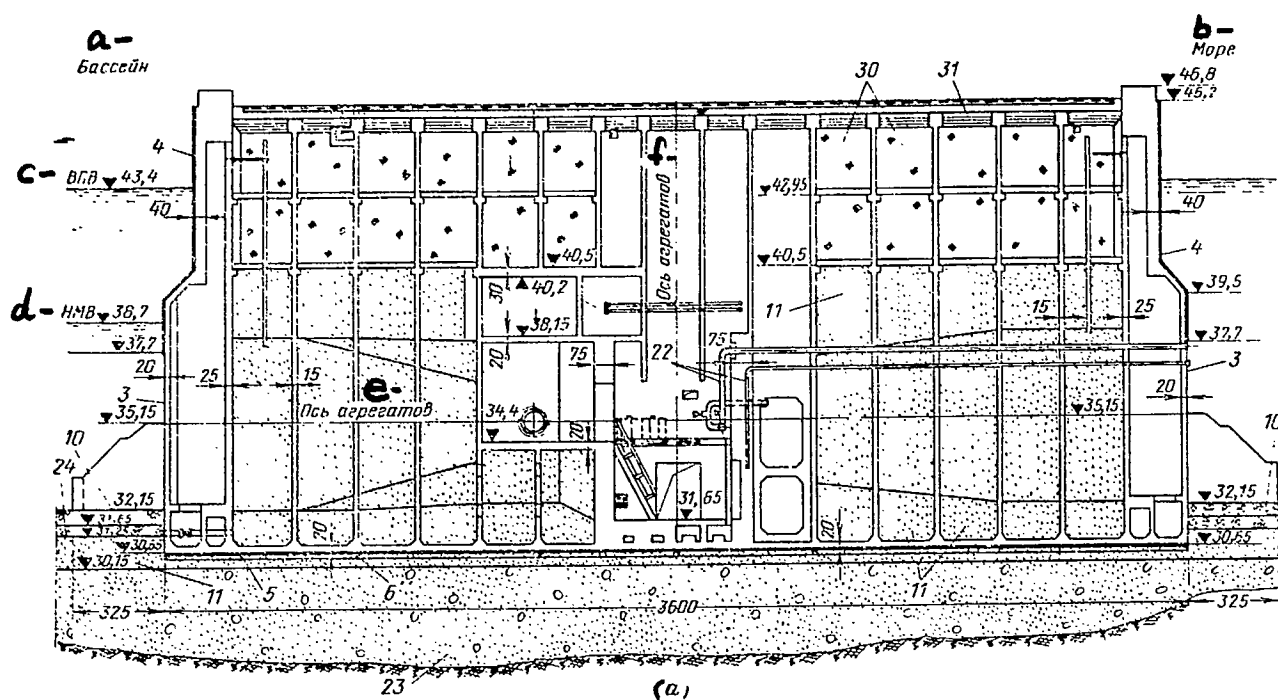
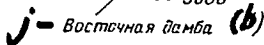


Figure 3-4. Transverse cross section along the block axis (a) and plan at the 43.50 meter mark (b).

1 - reversible bulb-type unit $N = 400$ kW, $n = 72$ revolutions per minute, $D_1 = 3.3$ meters; 2 - spillway; 3 - waterproofing; 4 - thermal insulation; 5 - acetate pads; 6 - anti-scour angular frame; 7 - cementation system; 8 - hermetic cover; 9 - access shaft; 10 - quarried stone support; 11 - sand-gravel mixture; 12 - open area for testing hydrothermal insulation coatings; 13 - derrick-gantry crane with a lifting power of 30/5 ton-force; 14 - control panel for the cathodic protection and control and gauge apparatus of the TPP; 15 - control panel; 16 - hydraulic drive system; 17 - conical metal conduit prior to installation of unit No. 2; 18 - oil pump assembly; 19 - oil system; 20 - ventilating system; 21 - conduit drainage system; 22 - pumping-out and drainage system; 23 - dense moraine deposits; 24 - transitional layer; 25 - quarried rock fill; 26 - loading shaft; 27 - grappling beam with a load carrying capacity of 30 ton-force; 28 - wooden thermal shield; 29 - sand-gravel mixture fill to the 40.5 meter mark, with hydrophobic soil above; 30 - hydrophobic soil; 31 - toothed concrete; 32 - spillway slab; 33 - stone-armored slopes $d \geq 500$ mm; 34 - stone-armored slopes $d \geq 900$ mm; a - basin; b - sea; c - higher high water; d - lower low water; e - axis of turbine units; f - axis of turbine units; g - west dam; h - axis of turbine unit No. 1; i - axis of turbine unit No. 2; j - east dam.



Обозначения 1-29 см. на рис. 3-2 и 3-3; 30 - гидрофобный грунт; 31 - шпиробой бетон; 32 - впадина подослива; 33 - крепление откосов камнем $d \geq 500$ мм; 34 - крепление откосов камнем $d \geq 900$ мм.

to bottom by 15 centimeter-thick bulkheads, but their central lower part is made thicker, forming walls for the inserts of the metal casings of the turbine water conduits (Fig. 3-2, a). The rest of the bulkheads each have two symmetrical openings in their lower portions for the installation of the turbine water conduits which, depending on the mode of operation, perform their function as either intake or suction pipes. The two pipes from the sea side, for a distance of 8.5 meters, are in the shape of a rectilinear truncated cone with $d_1 = 3.5$ meters and $d_2 = 5.64$ meters, sheared off further along on four sides by planes parallel to the axis of the cone and forming a rectangular cross section with rounded corners. The two pipes from the basin side have a complex cross sectional form, changing along their length from a 5.76 x 5.76 meter rectangle at the entrance to a cylinder ($D = 5.3$ meters) adjoining the thick bulkhead. Initially, the shape of the pipes from both the sea and basin sides was to have been a rectilinear truncated cone, but in the working drawings the cross section indicated above was used due to the requirements of the Neyrpic company, the turbine unit contractor. The minimum thickness of the suction pipe walls is 20 cm, the maximum is 45 cm.

The massive central bulkheads, to which lead the reinforced concrete conduits from both sides, have metal inserts made for the duct of the first turbine unit in the form of a casing forming the flow-through part of the turbine: two-way funnel-shaped openings (transitional cones) adjoin the runner wheel chamber. Along the duct of the second unit there is a one-way transitional metal cone connecting the two sections of the reinforced concrete turbine unit water conduits. The flanges of this transitional cone, which are bolted to support rings embedded in the concrete, will make it possible to replace the transitional cone with a flow-through section during the

installation of the second assembly. In this way, the transitional metal inserts connect the suction pipes from the sea and basin sides to form two individual turbine units water conduits.

As may be seen from the transverse section of the structure (Fig. 3-3, a), parallel to the side walls there are an additional four walls which bear up against the turbine water conduits and form, in the upper part of the structure, two outer and one central piers.

/50/

At the 40.10 meter mark above the first unit and at the 38.15 meter mark above the second unit the transverse bulwarks between the piers have cutouts which make it possible to span the bulwarks. At a height of 7.5-8 meters along the top of the bulwarks over the first turbine unit lies the spillway slab, while over the second unit lies the ceiling slab. In the sections of the block from the side of the sea and the basin, in the area of turbine unit No. 2, the slab shifts from the 38.15 meter mark to the 39.40 meter mark, where it forms, together with the central and end piers, a chamber where the control panels of the TPP, the monitoring and control equipment, the cathodic protection system, and the oil system are located. In the area of turbine unit No. 1 the slab shifts from the 38.15 meter mark to the 40.50 meter mark where, together with the central and west piers, it forms the surface spillway. In this section the slab is 20 centimeters thick, changing to a thickness of 30 centimeters above the chamber at the 38.15 meter mark. A hermetic hatch for the assembly and disassembly of turbine unit No. 1 is located on the surface spillway slab. The tank of the forced-oil system is located below it on the ceiling slab at the 38.15 meter mark. The connecting shaft to the bulb of the turbine unit is built into this slab (Fig. 3-2, a).

The control system and oil supply system chambers are divided into two stories by a metal ceiling (Fig. 3-2, b). These chambers are covered by a 25 centimeter-thick slab at the 46.00 meter mark. The slab rests freely on the piers in order to prevent it from binding during temperature deformations.

The tops of the piers have an enlarged section on which a floor slab rests at the 46.00 meter mark and in which the gantry crane track is embedded. In this slab, over turbine unit No. 2, there is a hatch for assembly and disassembly (Fig. 3-3, a). In the pier at the east side of the structure there is a small additional maintenance hatch at the 46.00 meter mark. Parts may be delivered to the 31.65 meter level through the hatches. A crane with a lifting power of 30/5 ton-force services all of the hatches. The entrance to the turbine unit block is located at the 46.00 meter mark and is constructed in the form of a vestibule.

All of the compartments are interconnected by stairs. There are two additional exits for emergency evacuation of the personnel at the 31.65 meter mark: in the east pier there is an exit at the 42.90 meter mark and in the west pier there is an exit to the adjoining dam.

/51/

The structural layout described above allows for a skeletal structure ensuring compact placement of the technical equipment, provision for its assembly and disassembly, convenience of operation, and high strength.

3-2. The underwater foundation area beneath the TPP structure

The construction of the foundation pit and the underwater foundation beneath the power plant structure without the use of cofferdams is a complex problem in the practice of hydroelectric power plant construction. For this reason, in addition to achieving a concrete solution for the Kislaya Guba TPP, it was necessary to accumulate experience for the future construction of

foundations by this method under conditions at the sites of powerful TPP's and establishing the equipment requirements for underwater excavation in tidal seas. It was no less difficult to overcome the skepticism, and uncertainty of the possibility of successfully carrying out these operations, of a number of experts, including the main subcontracting organization (Expeditionary Unit of the Emergency Rescue and Underwater Technical Operations [ASPTR] of the Murmansk Steamship Company) although, getting ahead of the story, it should be noted that, in the end, the ASPTR did a complete and first-rate job on this task¹.

Even during the initial surveys the very favorable configuration of the Kislava Guba channel made it possible to designate several site variants. After carrying out the explorations, testing on a model, and also economic evaluation of the variants, the main TPP site was chosen, and the contours of the foundation area (Fig. 3-5) were determined, with a minimum width of 35 meters, a maximum width of 65 meters, and an average excavation depth of 5 meters.

The elevation of the base of the foundation area is determined by the diameter of the turbine, the configuration of the suction pipes and the level of the lowest low water (38.70 meters), as well as by the thickness of the artificial bed beneath the block and the underwater supports. The joint of the foundation area with the natural surface has a longitudinal slope of 1:5 and a transverse slope of from 1:3 to 1:2 which, according to the research of the All-Union Scientific Research Institute of Hydraulic Engineering, ensures the stability of these slopes. The plan dimensions of the foundation pit are determined by the dimensions of the block plus a certain margin for submersion, as well as by the possibility of installing the equipment for the excavation of the pit.

¹ ASPTR Chief Yu. N. Martyshkin and Chief Engineer V.S. Volovikov were in charge of the operations.

layer is 1-1.5 meters on the west bank side of the axis and 3-4 meters on the east bank side. To the north of the transverse axis of the foundation area (sea) the granites are located below the designated excavation line, as are the granites along the longitudinal axis of the block to the south of the transverse axis of the foundation area (basin) in a 15-17 meter-wide zone. Remember that the top of the layer of rubbly deposits is at the 36.0 meter mark (level of maximum high tide 43.40 meters, theoretical null depth 39.45 meters).

/53/

Table 3-1

Classification of the regolith underlying the excavation of the underwater foundation in term of Construction Norms and Regulations SNIp categories.

Brief Description	SNIp Categories	Volume, ³ thousand m
Marine deposits:		
fine-grained and inequigranular sands with gravel, rubble and boulders, coquina with sand	II	7.5
rubbly gravel with inequigranular and fine-grained sand	IV	11.5
Rocky regolith of the slopes of the foundation area: fissured granite	VI	6.3
Rocky regolith at the bottom of the foundation area: granite	VIII	0.5
TOTAL		25.8

The total volume of operations for excavating the foundation area was initially estimated at 22.0 thousand m^3 , subsequently, as a consequence of a change in the method of loosening the ground and the need for greater width for the passage of the floating crane along the block, the volume of the excavated ground was increased and comprised 25.8 thousand m^3 (Table 3-1). The increased volume consisted mainly of fissured rocks.

In choosing the machines for preparing the foundation area the determining factor was the geological structure of the site but, naturally, the dimensions and configuration of the Kislaya Guba channel as well as the actual industrial capabilities of the subcontracting organizations and the target dates for completion of the excavation of the foundation area were also taken into account. The possibility of involving in the excavating operations both dredging equipment and floating cranes was examined but, due to considerations explained below, preference was given to the later.

The small excavation volume disallowed the use of machines with a high productive capacity. The use of a dredger was also impossible due to the significant content of boulders in the soft regolith (30%) and the large proportion of rocks (6.0 thousand m^3) in the excavation. In addition, the width of the channel in the area of the site restricted the maneuverability of the dredger and the scows used for removing the earth and, as a consequence of the outcrop of rocks along the banks, there arose the danger of frequent damage to the dredges and even the dredge frames. The need for significant loosening of the ground combined with the possibility of loosening only to a shallow depth would have resulted in idle time for the floating equipment. /54/

In modern practice the most efficient mechanical means for carrying out underwater excavations under complex geological conditions are considered

to be single-bucket and clamshell excavators. There were several single-bucket excavators in the Soviet fleet of dredging mechanisms, but the Northern Basin did not have them at its disposal and there was no real possibility of obtaining them by the start of operations at Kislaya Guba. For this reason the excavation plan at Kislaya Guba called for a floating crane with an assortment of clamshell equipment of various sizes and weights which, in fact, was used during construction. The required capacity of the clamshell buckets was determined for the conditions at the site on the basis of the optimum ratios between the capacity of the clamshell and its weight under hard digging conditions.

Clamshell capacity, m ³		1 - 1.5	2	3
Clamshell capacity, t		6	8	12-14

On the basis of the weight of the clamshell and the dimensions of the boulder inclusions it was deemed suitable to use a floating crane of 10-15 ton-force. In terms of lifting capacity these requirements were satisfied by a non-self-propelled floating crane with a lifting capacity of 10 ton-force of the "Valmet" type which, thanks to the Ministry of the Maritime Fleet of the USSR, was delivered to Kislaya Guba from Lake Onega. The use of this type of floating crane for excavation under the given conditions, however, was not an optimum solution because its construction did not provide for the free fall of the clamshell during its descent to the ground. The clamshells were of low weight with high capacity. In the open position the central block part is at the level of the ends of the jaws, which hinders the jaws from digging into the ground under the weight of the clamshell. For this reason the choice of the "Valmet" crane was dictated solely on the basis of its availability.

A barge with a carrying capacity of 100 ton-force and with a drop bottom was used to transport the excavated earth. A seagoing diving boat was used to tow the barge while at the same time taking care of all the diving operations connected with the excavation and loosening of the ground.

/55/

In the natural state (prior to construction) the incoming and outgoing tidal currents in the Kislaya Guba channel reached speeds of 3.5 meters per second. For this reason, in carrying out the operations of underwater excavation, preparation of the foundation, submersion of the TPP block at the site, and construction of the connecting structures, provision was made for isolating the inlet by constructing a permeable embankment in the narrowest part of the channel (length of embankment along the top - 40 meters, height 7-8 meters, width along the top - 8 meters, total volume of embankment - 6.5 thousand m^3). The permeability of the embankment, which was formed without fine sand and clay fractions, was required by the necessity of passing through it the runoff from the fresh water catchment area, whose discharge during freshets is as high as 3 m^3 per second.

The embankment (Fig. 3-6) was designed in such a way as to provide for the permissible slopes and for a large enough top width to permit driving along it during construction and dismantlement. During construction of the embankment consideration was taken of the specifics of the tidal flow, which differs from river flow by its reversible current, with a stop during the alternation of the tides every 6 hours 12 minutes. It was foreseen in the project that with increasing constriction of the channel its throughput capacity would decrease, causing an increase in pressure differentials, which would result in increasing speeds of flow. For this reason the damming was calculated in such a way¹ that the final stage, i.e. completion of the

/56/

¹Damming project and computation carried out by engineer Yu. A. Fishman.

embankment, would take place during the period of the smallest tidal ranges (small quadrature). Inasmuch as the embankment was to be constructed over a period of 20 days, the start of construction fell during the period of high syzygial levels. In order to determine the stone size, a transverse section of the embankment was divided into three parts corresponding to the three constrictions of the flow. For each constriction the maximum pressure differential was computed and from it the maximum speed was determined, after which the stone size $D_{\max}$ was calculated by the well known formula $D_{\max} =$

$$\frac{v_{\max}^2}{2v^2 g \frac{\gamma_s - \gamma_w}{\gamma_s}}$$

where $v_{\max}$ - is the maximum speed in the section in the presence of the given constriction; v - is the coefficient of stability of the stone; γ_s and γ_w - are the specific weights of the stone and water respectively; g - is the acceleration of free fall.

Hydraulic computation of the embankment, taking into account the constriction of the channel and stoppage of the current during low and high water, showed the possibility of constructing the dam using stones 0.22 - 0.45 meters in size, with the need to increase the size of the stones being dumped into the standing water to 0.5-0.6 meters only during closure.

3-3. Connection of the block with the foundation, races and banks

a) Underwater profile.

During the design process the construction of the foundation, and the underwater profile associated with it, were changed several times. The reason for these changes was the complexity of the problem, being tackled for the first time, of constructing a TPP structure by the floating method, as well as more accurate information about the geological structure of the area (an increase in the bedding depth of the bedrock by several meters), and evaluation of the permeability and scouring characteristics of the artificial foundation.

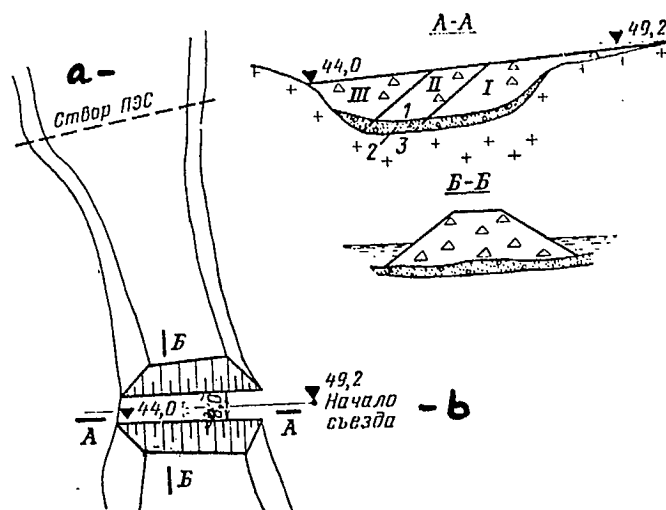


Рис. 3-6. Банкет.

1 — каменная отсыпка; 2 — морские отложения; 3 — граниты; I, II, III — части банкета, соответствующие сужению потока.

Figure 3-6. Embankment.

1 - riprap; 2 - marine deposits; 3 - granites; I, II, III - parts of the embankment corresponding to the narrowing of the stream; a - TPP site; b - beginning of incline.

On the basis of geological (1938-1939) and geophysical (1960-1962) surveys, which established the presence of granite gneiss at the TPP foundation level, the following variant of the underwater foundation was adopted in the design program: a 75 centimeter-thick gravel bed, with the possibility of installing a cemented cut-off formed through the floor slab of the floating block. This variant required the development of a method of joining the block with the cut-off without the application of concentrated forces, which would be very undesirable with a thin floor slab (this was done in the project of the Lumbovka TPP) [14, 15].

In conjunction with the above a variant was proposed for levelling the surface of the rock by means of underwater concreting, but in view of the fact that the realization of underwater concreting with a high degree of precision was not deemed possible, while the presence of even small irregularities could result in a disruption of the integrity of the block, this variant was not adopted.

Clarification of the geological structure of the area revealed that almost the entire foundation area of the block was composed of inequigranular marine deposits with older inclusions, and that rock outcrops are of a local nature. In addition, the permeability coefficient of the poured gravel is tens of times greater than the permeability coefficient of the natural bottom. This could have led to the erosion of sand along the ground contact plane in the foundation. For this reason it was proposed to replace the gravel bed beneath the block with a 50 cm-thick sand bed. This also permitted more precise levelling. In the approach channels it was proposed to install a triple-layered reversible filter, which would impede the erosion of the foundation soils, instead of riprap¹. In order to obtain a reliable contact with the foundation it was proposed to fit the block with blades² which would bury themselves in the sand layer.

These recommendations were reflected in the decision of the Technical Council of the Ministry of Power Engineering of the USSR, which approved the TPP project in 1963.

During the amendment of the foundation design from the point of view of ensuring its resistance to scouring, instead of blades it was initially decided to use a precast reinforced concrete frame with rubber packing, to be installed in the artificial foundation. Subsequently however,

¹ Proposed by Doctor of Technical Sciences, Professor N.N. Verigin.

² Proposed by Doctor of Geological and Mineralogical Sciences, M.N. Bindeman.

when it became evident that installation of the frame would be quite difficult, upon the recommendation of the All-Union Scientific Research Institute of Hydraulic Engineering, the previously proposed metal blades were accepted with a system of cementation as a back up. Cementation was to have been carried out prior to the plant going into operation only if voids were discovered beneath the floor slab of the block.

On the basis of laboratory work carried out at the All-Union Scientific Research Institute of Hydraulic Engineering¹, the sand bed was replaced with sand and gravel bed, changing the approach channels to a 1.1 meter-thick sand and gravel layer with a double strengthening layer of gravel (crushed stone) and stone placed on top. The gravel and stone strengthening layer is 0.9 meters thick. The rejection of the reversible filter was based on studies which showed that, with time, a natural reversible filter from on the surface of the sand and gravel mixture, which prevents further erosion.

Sand and gravel from the Ura-Guba quarry, with a sifting of fractions larger than 50 millimeters were recommended for fill in the foundation. The soils are inequigranular with a coefficient of inequigranularity of 10, in some samples 20 or greater. The content of sand fractions smaller than 2.0 millimeters was 15-35%. The contact of the soils with the bottom of the block and with the marine deposits is resistant to scouring at a pressure gradient of 10. The mean value of the coefficient of permeability is 60 meters per day. The soils satisfied the requirements for stability against erosion by the percolating stream passing through the riprap. These same soils were used in the knife edge supports and in the beds of the approach channels, but in these areas it is permitted to remove boulders only larger than 300 millimeters.

¹The work was carried out in the filtration laboratory of engineer V.D. Zhebrovskaya under the direction of Candidate of Technical Sciences E.A. Lubochkova.

Laboratory studies of the filtration regime using electrohydrodynamic models (spatial problem on a liquid model), which were carried out during the design period of the TPP and which included a determination of the filtration pattern, the plotting of lines of equal pressures in the foundation, and the determination of average and local pressure gradients, were confirmed by actual data from eight tubular piezometers installed in the foundation along the east and west sides at a distance of 1 meter from the wall (Fig. 3-7).

/59/

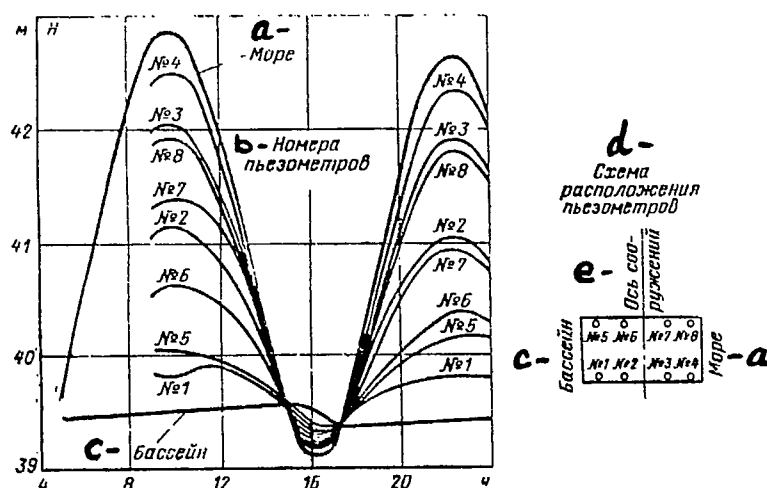


Рис. 3-7. График изменения уровней воды в пьезометрах в зависимости от уровней воды в море и бассейне.

Figure 3-7. Graph of water level changes in the piezometers as a function of the water levels in the sea and in the basin.

a - sea; b - piezometer number; c - basin; d - piezometer distribution scheme; e - axis of structures.

Observations carried out in 1969 made it possible to establish that changes in water level in the sea were reflected in the piezometer readings. The piezometers regularly show a decrease in levels in the direction of the low water level in the races. During observations the pressure gradients fluctuated within the limits of 0.08 - 0.15 while the percolation beneath the block was 0.12 - 0.15 m³ per second. These observations revealed that there are no stagnant zones within the block foundation which would not show the effect of changes in the pressure or direction of the percolating flow, that there are no "lag" zones, i.e. zones in which the direction of the percolating flow would be opposite to the main flow.

b) Connecting dams (design and execution)

The connections with the banks which, together with the block, cut off the flow (Fig. 3-1), are formed by a wall composed of ShK-1 sheet piles, installed through the sand knife edge supports, down to the bedrock or stone embankment composed of quarried rock of various fractions. The facing is applied over the gravel layer, which is placed over the pioneer fill on both sides of the wall. The connecting dam has a trapezoidal shape 8 meters wide at the top, with the slopes laid at 1:1.3, and a height of 15 meters. In the zone of wave action the slopes of the dams are reinforced on the north side with rocks larger than 900 millimeters, on the south side they are reinforced with rocks larger than 500 millimeters. This reinforcement is calculated for the action on the slope of waves estimated at 3 meters from the sea side and 1 meter from the basin side.

The erection of the dam was begun in September 1968, as specified in the project, with the placement of pioneer fill and gravel reinforcement with the aid of a floating crane. This same crane was used to place the Larsen-5 guiding sheet piles which connect the block with the banks. The emplacement of the sheet pile, which was begun from the east bank, was accomplished using a VPP-1 vibrator. The facing of the west and east dams was accomplished by the pioneer method from the east bank with the aid of a MAZ-205 automatic dumper. Temporary decking was installed on the block in order to apply facing to the west dam. During the final stage (end of December 1968), in order to speed up the start-up of the plant, a floating crane was used which moved the rock, delivered by the dumpers, from the west side of the block to the bank. In 1969, after the start up of the TPP, the slopes were reinforced with large stones.

c) Reinforcement of the approach channels (design and execution)

The most critical joining element is the upper reinforcing layer of the beds of the races (Fig. 3-2), on which depends the integrity of the underlying layers. For this reason in 1964-1965, at the All-Union Scientific Research Institute of Hydraulic Engineering¹, using a 1:50 scale model, hydraulic research was carried out for choosing the type of reinforcement of the approach channels at the Kislaya Guba TPP. Eleven characteristic operational regimes of the plant were studied, with discharges from 100-160 m³ per second and levels in the range of 40.0-42.5 meters. Evaluation of the regimes of TPP operation was performed according to the magnitudes of the maximum mean speeds in the races at distances of 30-50 centimeters from the bottom (full scale) depending on the type of joining of the races and

¹The work was carried out in the Laboratory of River Hydraulics by Candidate of Technical Sciences V.A. Solnyshkov under the direction of Doctor of Technical Sciences F.G. Gun'ko.

degree of displacement of the flow toward the banks. The results of the laboratory tests showed that, under all of the operational regimes examined, the bottom speeds did not exceed 5-6 meters per second, during which a bottom type of interconnection was observed with maximum speeds in the region of flow right at the bottom. During the first stage of the tests the size of the stable stones used for reinforcement of the races was determined on the basis of the empirical formula

$$d_H = 1,4 \frac{v_0^3}{\sqrt{H}},$$

where d_H is the average size of a stable stone in centimeters; v_0 is the speed of flow at the bottom in meters per second; H is the depth of the flow in meters.

This formula was confirmed by tests on the model, for which reason, on the basis of the operational regimes studied, the size of the stones was set at 50-60 centimeters. In order to facilitate reinforcement however, it was thereafter decided to use 25-50 centimeter stones, which brings down costs during operation. During a maximum discharge through the turbine units of 50 m³ per second the total discharge through the gates should not exceed 100 m³ per second. This is based on the practical requirements of the plant during operation. In order to avoid destruction of the reinforcing layer during testing of the TPP structure under critical regimes (during testing of the dynamic strength of the structure) it was proposed, in order to dampen the energy, to use the embankment located in the race on the basin side, the partial erosion of which was to occur during testing.

On the basis of All-Union Scientific Research Institute of Hydrological Engineering recommendations, it was decided that a 40 centimeter-thick crushed stone or gravel layer, with a fraction size up to 150 millimeters, would be spread in the approach areas over the 110 centimeter-thick

layer of mixed sand and gravel. Over this layer would be laid a 50 centimeter-thick weighting layer of stone. All three layers would extend horizontally to the joint with the natural surface.

On the basis of this design solution, after the ballasting of the block, the races began to be filled to the specified marks with a sand and gravel mixture. A modification of the initial decision was that the basin part of the joint between the foundation and the race was filled with gravel and reinforcing stone, with the aid of a floating crane, after installation of the block. In order to accomplish this the crane was brought into the basin several times through the passage between the west bank and the block. Into this same area a scow with fill was towed with the aid of the crane capstan, which was secured to the side of a crane pontoon, and fill was spread with rough (± 15 centimeters) levelling accomplished by divers. Following the divers' orders, the crane was used to either release more fill or rearrange it. Work on reinforcing the races on the basin side was completed on 20 November 1968. /62/

However, due to the fact that testing of the TPP structure under critical regimes was necessarily conducted with the embankment disassembled, the reinforcement on the basin side was partially destroyed. This destruction led to the erosion of the foundation to a depth of 2 meters along the center of the block, with the washed-out hole extending as far as the undermining protection blade to a width of 1.5 meters, and also to the formation of a washed-out area of approximately 10 m^2 in front of the TPP structure at a distance of 5 meters from the bottom weir.

In order to eliminate this erosion, the races were additionally reinforced in the first half of 1969. The reinforcement restoration repair work was made more difficult by the fact that the approach channel from the basin side was cut off from the sea by a closed pressure front which prevented access by floating equipment. For this reason it was proposed that a maintenance crane be used. The sand-gravel and gravel (crushed stone) was transferred from dump trucks into a special bucket after which it was delivered by crane to the underwater maintenance operations site. These maintenance operations were very time consuming and quite complex due to the restricted radius of operation of the crane beam. Divers indicated the location at which each bucketload delivered by the crane was to go underwater and then manhandled the reinforcing material to its assigned underwater location. In order to significantly decrease the volume of this laborious work it was proposed to not restore all of the destroyed reinforcing layer completely but to limit the repair only to the installation of a small "gasket" or "plug" in the form of a reversible filter only where it adjoined the gate itself. It was proposed not to add fill in eroded areas where the bedrock was exposed and, in areas covered by large stones, where the flow had apparently already formed an erosion-resistant bed, to only add additional larger stones. This proposal was examined and approved by the All-Union Scientific Research Institute of Hydraulic Engineering and was fully implemented. /63/

In the summer of 1969 (July-September) the operations were extended with the participation of a group of divers from the Moscow Aqualung Club of the Moscow Institute of Aviation. Additional reinforcement was carried out at the same time on the approach channel from the sea side. This was done, as during initial reinforcement, from a barge, but it was loaded at its berth with stone delivered from shore. The ashlar was unloaded in place as indicated by the diver.

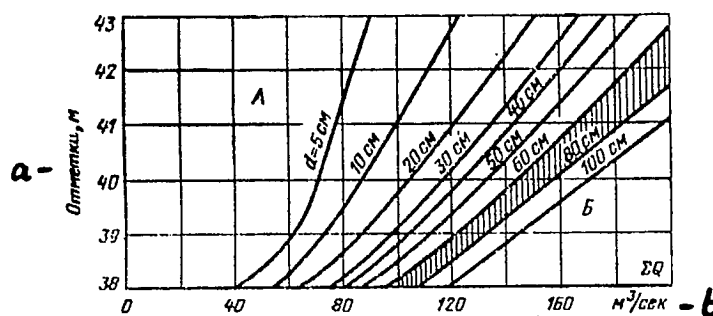


Рис. 3-8. Зоны разрешенных режимов работы ПЭС для различной крупности крепления.
А — зона без размывов; Б — зона размывов.

Figure 3-8. Zones of permissible TPP operating regimes for various sizes of the reinforcing layer.

A - zone without erosion; B - erosion zone; a - levels, meters; b - m^3/second

After the completion of each phase of reinforcement, water was passed through with the turbine, spillway and weir operating in various combinations. Careful underwater inspections were then carried out, including measurement and underwater photographs of characteristic profiles. When necessary, as indicated by these underwater inspections, additional reinforcement was carried out.

The results of allowing the water to flow through, as plotted on the graph of the relationship of the levels in the tail race with the total discharges for different ($d = 5, 10, 20$ centimeters, etc) sizes of reinforcing stones (graph plotted from the results of summation of data obtained at the All-Union Scientific Research Institute of Hydrological Engineering using the erosion model and at the site), showed close agreement of data obtained at the site with those of the model tests, i.e. the erosion which occurred was within the erosion zone (Fig. 3-8). Exceptions were certain points corresponding to erosion at the site but lying within the no-erosion zone on the graph. Analysis of these throughputs of water made it possible

to establish that these points correspond to regimes characterized by acute dislocation of the flow (throughput only through the bottom weir) or the concentrated fall of water from a spillway.

Inspection of reinforcement with a stone size of 50-60 centimeters, which was recommended by the All-Union Scientific Research Institute of Hydrological Engineering, revealed that erosion of it was insignificant. For this reason, in order to remove restrictions during the majority of operational and experimental regimes in the joint sections between the block and the reinforcement, the size of the stones was increased to 1 meter. Even under these circumstances, however, the unfavorable regimes indicated above must be excluded or corresponding costs curtailed.

It can be concluded from the results of the laboratory and on-site tests which were carried out that, under conditions of TPP construction by the floating method, reinforcement of the races with riprap in the underwater excavation is expedient and reliable. The dimensions of the fill rocks, with relatively even distribution of per-unit costs along the spillway front, may be calculated by the formula proposed by the All-Union Scientific Research Institute of Hydrological Engineering. The actual work of forming the joint, however, is laborious, requiring constant underwater control. As a result, for large scale construction, it would apparently be expedient to install prefabricated reinforcing elements, especially at the juncture with the block.

The volume of earth used was $3,060 \text{ m}^3$ of sand and gravel fill, $1,250 \text{ m}^3$ of gravel reinforcement and $11,200 \text{ m}^3$ of stone fill and reinforcement. During the repair operations an additional 37 m^3 of sand-gravel and 25 m^3 of crushed stone was dumped and 497 m^3 of stone was laid.

Individual costs for the above work on the project proved to correspond to costs in the Murmansk region and came to: 6.70 rubles for 1 m³ sand-gravel fill, 9.17 rubles for gravel, 11.04 rubles for 1 m³ of stone fill and reinforcement; 1.61 rubles for rough levelling of 1 m² of sand-gravel and gravel fill; 9.25 rubles for meticulous levelling of 1 m² of sand-gravel fill; 3.01 rubles for levelling 1 m³ of rock fill. The cost of materials is included in these unit costs for fill and the placement of stones.

Unit costs for additional work, after testing of the structure under critical regime and throughput of water in order to establish restrictions during experimental operation, proved to be significantly higher than usual due to the limited use of machines and floating equipment and the relatively small volume of underwater diving operations.

Chapter 4

POWER ENGINEERING AND EQUIPMENT

4-1. Power Engineering

a) Selection of installed capacity and number of turbine units.

The TPP capacity which corresponds to maximum utilization of the energy potential of Kislaya Guba is determined by the formula [3]

$$N_{\text{installed}} = 250 A_{\text{mean}}^2 F = 1,500 \text{ kW} \quad (4-1)$$

(where $A_{\text{mean}} = 2.37$ meters and $F = 1.14 \text{ km}^2$).

This capacity under the given conditions (width and depth of the channel at the chosen site) may be obtained by installing four bulb-type units and a runner wheel with a diameter of 3.3 meters. Analysis of the problem [3], however, shows that tidal power plants, whose construction requires the damming off of arms of the sea and consequently involves significant difficulties and the outlay of many resources, may be economically justified only if the installation is large. This is also required by the necessity and expediency of using tidal energy in unified power grids which cover large economic regions. Even when the Kislaya Guba TPP was first being planned it was already clear that the cost of this small installation, which is of experimental importance, would be high. Under these circumstances, of course, the capacity and number of units would have to be determined by the conditions of the experiment itself.

For this reason it was decided not to install three units, as was proposed in the first (prewar) project [8] and the 1961 project [3], which were to have a TPP with one French unit and one Soviet unit in a twin-shaft design (two turbines, with a Hook's joint and one generator). Since it became evident that it would be impossible to reliably execute such a combination for the TPP units industrially (5 - 15 megawatts) it became necessary to reject this variant and confine construction to two units for the experimental TPP (1 French, the second of modified Soviet construction).

According to the project, the Kislaya Guba TPP is to work under regimes representative of optimum utilization of tidal energy: at maximum efficiency (at the base of the load curve) and in peak regime, as required by the system (during the two hours in winter when the system is under greatest load, from 4-6 pm in order to cover the peak). During this process, 6-cycle operation of the tidal hydroturbine units must take place (reversible turbine and pump operation and reversible idle throughput). In accordance with the capabilities of the unit manufactured by the Neyrpic Company these operations are accomplished at heads of 0.18 - 2.5 meters (capacity at the design head of 1.28 meters is 400 kW), while under the reverse turbine regime the head which produces 400 kW is 1.65 meters. Direct throughput, during which the units are kept at a standstill, is accomplished at heads of 0-1.28 meters and reverse throughput at heads up to 1.60 meters.

In view of the fact that the mean non-regulated tidal discharge (ebb) at the TPP site is 105.5 m^3 per second (350 m^3 per second maximum), while one unit can pass through only 50 m^3 per second at maximum head, in order to increase utilization of the current there was provision made that, prior to the installation of the second unit, its conduit together with the upper spillway, would serve as an opening for the throughput of water and thus speed up the equalization of levels during the switchover of the turbine to reverse head. In this manner, the bottom weir will make it possible to increase energy output by the first unit.

b) The TPP operating regime and its optimization

A characteristic feature of TPP regimes is the fact that even a relatively small deviation of the regime from the optimum results in significant regime losses. The use of approximation methods of calculation for selecting the regimes of a TPP causes a significant decrease in the power

engineering and fuel efficiency of its operation. In a number of cases these losses are in the tens of per cent. This situation required the development of a special method for computing TPP regimes using modern methods of nonlinear mathematical programming and computers.

/67/

The initial data required for establishing the optimum TPP regimes are: the pattern of changes in the water level of the sea in time throughout the period of regulation (Fig. 2-4), the operational characteristics of the TPP unit for all possible operating regimes (Figs. 4-6 and 4-7), the characteristics of the throughput capacity of the water passages and the turbine conduit, and the topographical characteristics of the tidal basin (Fig. 2-6).

The problem of optimizing the TPP regime is formulated in the following manner. Water levels in the TPP basin and all initial characteristics are set for the beginning z_0 and end z_n of the period of computation T . What is required is to determine for computational period T a curve of changes in the water level in the basin over time $z(t)$, such that it passes through the set cut-off levels and ensures the realization of the adopted criterion of optimality (the criterion of minimum operational expenses by the power grid caused by the TPP regime).

During this process restrictions on the regime must be observed in the following areas: on the water level in the basin, on the throughput capacity of the TPP units for all regimes, on the throughput capacity of the water passages and turbine conduit, on the minimum and maximum heads in the turbine regimes, on the maximum lift and pumping-out depth for pump regimes, on maximum heads during which the operation of the water passages and the turbine conduit is permissible, on the minimum permissible generating capacity of the TPP, and the maximum permissible intake of the TPP.

The last two restrictions appear when the TPP is required to cover an assigned part of the load curve peak.

Analysis revealed the insurmountable difficulty of solving this problem by the methods of classical calculus of variations, as was done for the simplest case of a unidirectional TPP by R. Gibrat [26].

This is due to the difficulties of calculating the limits and the impossibility of formulating conditions of transversality (conditions of interdependence of the extremes at transition points, which appear during switchovers of TPP operating regimes). For this reason modern methods of nonlinear mathematical programming were called upon for the solution of this problem. /68/

The problem of determining the most efficient operating regime for the electric power plants of an energy grid which does not contain a TPP is in itself extremely complex. The presence of a tidal power plant in the energy grid makes this problem immeasurably more complex because the basic method for calculating the most efficient operating regime for a power plant in a power grid, the method of incremental rates, is inapplicable for a TPP.

In order to curtail the computation time an iterative approach¹ was suggested, in which the optimum regimes of the power grid and the TPP are determined by separate calculations during each iteration, but the series of succeeding iterations takes into account the interdependence of the regimes of the TPP and the power grid. In order to do this the concept of the coefficient of worth $K(t)$ of a kilowatt of TPP capacity at a given instant of time is introduced. Numerically this coefficient is equal to the magnitude of the relative increment in the power grid (expressed as fuel or cost) in the node to which the TPP belongs at its given capacity curve. This coefficient may be calculated by any known method for computing the optimum regime of a power grid which includes only traditional power plants.

¹This work was carried out at the Laboratory of Theoretical Research on Power Grids, All-Union Scientific Research Institute of Electric Power Engineering, by Candidate of Technical Sciences V.N. Silakov under the direction of Doctor of Technical Sciences N.A. Kartvelishvili.

In order to reduce the variational problem to a problem of nonlinear mathematical programming the computational period T is divided into n intervals of length Δt . The operating regime of the TPP in any i -th computational interval is fully defined by specifying the levels of water in the basin at the beginning z_{i-1} and end z_i of this interval if all its internal variables average out. The equation of optimality is written as:

$$\sum_{i=1}^n K_i N(z_{i-1}, z_i) \rightarrow \max, i = 1, 2, \dots, n.$$

In this way, the variational problem is reduced to a problem of optimization of the function (n-1) of the variables: $z_1, z_2, \dots, z_{n-1}$. The number of variables per unit is less than the number of intervals because /69/ the level of water in the basin at the end of period T is specified by the conditions of the problem.

In order to solve the equation of optimality (4-2) the method of differential dynamic programming was used. This method is local, iterative and consists in sequential improvement of the regime of the initial approximation, i.e. it guarantees the determination of the best regime only when the target function has a single extreme.

If the TPP is not absolutely required to take part at the peak of the power grid curve the target function has a single extreme. In those cases when the TPP must participate at a given capacity in covering the load curve peak, the problem of optimizing the TPP regime becomes multi-extremal. The reason for the multi-extremality is not the nature of the target function but rather the disconnectedness of its domain of definition.

A special method was developed which permits the determination, without sorting through all of the sub-domains of the domain of definition of the target function, in which sub-domains there are peak TPP regimes

which may be practically realized. The global extreme in this case is determined in the following way. The first approximation regime is defined in each of the obtained sub-domains. Local extremes from the various first approximation regimes are then found by the method of differential dynamic programming. The regime which will be best in accordance with the accepted criterion of optimality is chosen by comparison from the obtained local regimes.

Computer calculations for a representative tidal period (29.5 days) produced the TPP operation curve in base regime (for maximum output) shown in Figure 4-1.

Analysis of this curve makes it possible to draw the following conclusions. Switchover to strictly one-way operation is found to be advantageous only during two days out of 28 (the 20th and 22nd). At other times the TPP operates as a two-way installation.

During syzygial tidal amplitudes the TPP output in reverse turbine regime is only slightly higher than in direct turbine regime. With decreasing tidal amplitudes an increasingly greater portion of the output is produced by the direct turbine regime. This is explained by the greater efficiency of the unit in direct turbine regime. In addition, operation under direct turbine regime under equal head is always more efficient because equal changes in the level of the basin under direct turbine regime result in higher discharge values than under reverse turbine regime because the TPP basin has a trapezoidal shape. The small throughput capacity of the unit under turbine regimes results in the fact that during syzygial tides the waiting periods are curtailed and do not exceed 30 minutes. During the highest tides the waiting periods practically disappear and the turbines are switched on as soon as minimum head is attained.

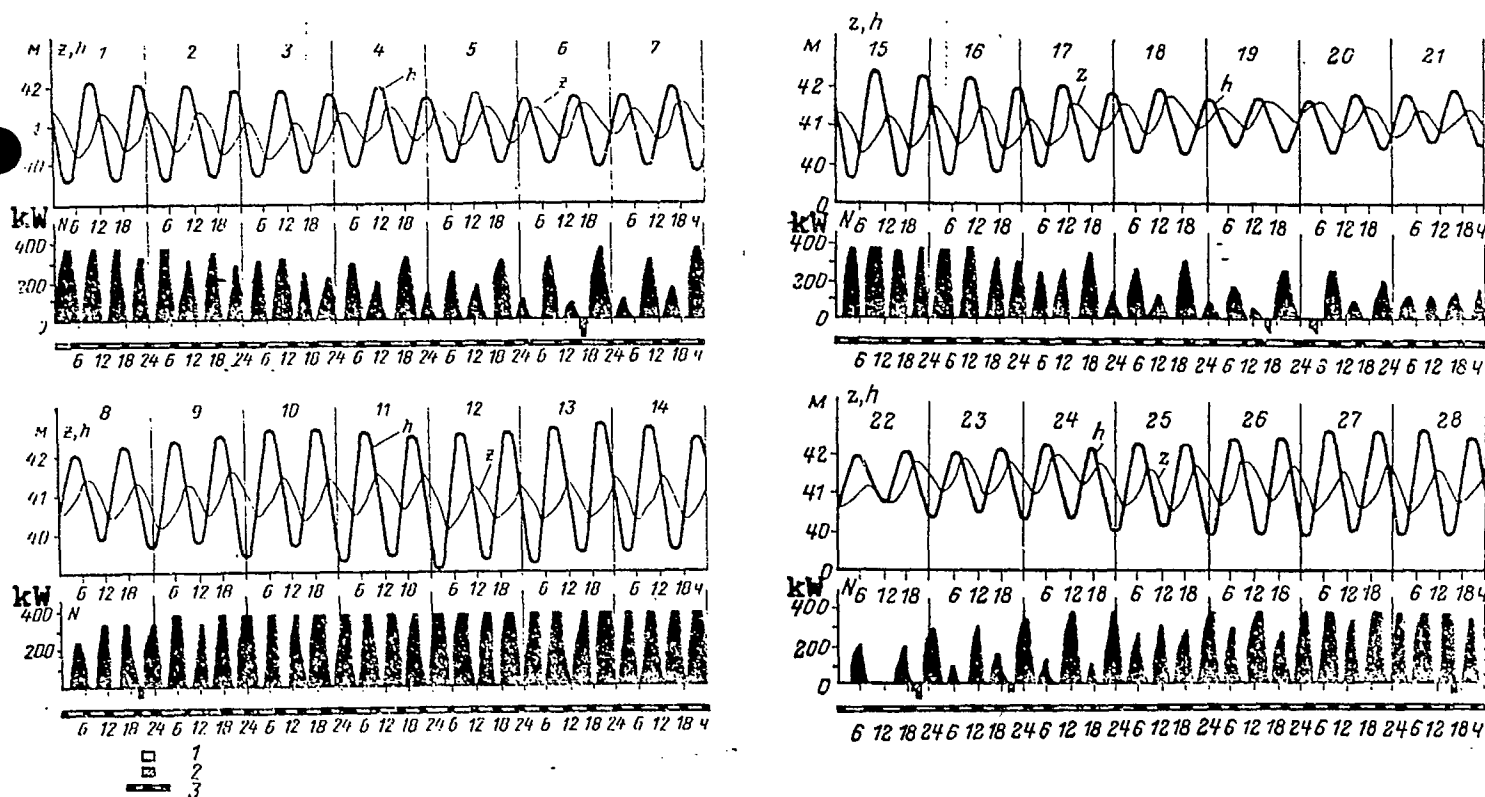


Рис. 4-1. Оптимальный базисный режим Кислогубской ПЭС (сутки 1—28).
1 — в турбинном режиме; 2 — в насосном режиме; 3 — периоды, в которые производится холостой сброс через подслывы

Figure 4-1. Optimum base regime for the Kislaya Guba TPP (days 1-28).

1 - in turbine regime; 2 - in pumping regime; 3 - periods of idle discharge through the spillways.

The use of pumping regimes in order to increase the general output of the installation is expedient only during the lowest tides at quadrature when the TPP operates practically as a one-way installation. The duration of pump operation and their capacity are practically insignificant. This makes possible the recommendation that pumping regimes not be used at all during base level operation of the TPP.

/71/

Idle discharge through the bottom weirs and spillways is performed during every cycle. As a rule, this idle discharge is begun while the turbines are still operating. During most days of the computational period, idle discharge through the water orifices is begun shortly after the moment of high or low water and continues until equalization of the levels in the basin and the sea. An exception are the periods of tides at quadrature when the TPP switches over to one-way operation.

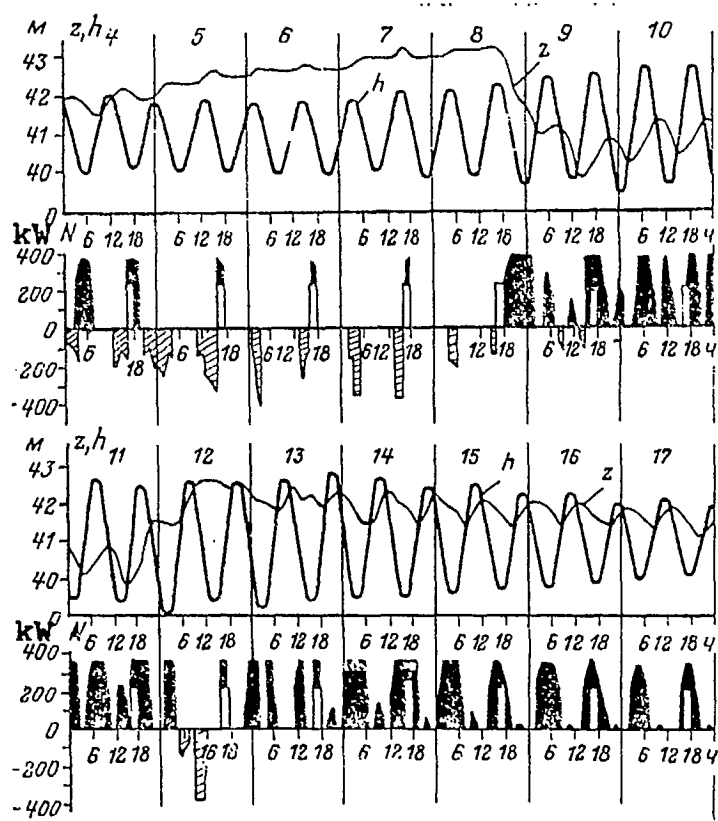


Рис. 4-2. Оптимальный пиковый режим Кислогубской ПЭС
($N=220$ кВт, сутки 4—17).
Обозначения 1—3 см. на рис. 4-1.

Figure 4-2. Optimum peak regime for the Kislaya Guba TPP

($N = 220$ kW, days 4-17).

1 - in turbine regime; 2 - in pumping regime; 3 - periods of idle discharge through the spillways.

Figure 4-2 shows the optimum peak regime for operation of the TPP over a 14-day period when the guaranteed peak capacity is equal to the displacement capacity, i.e. 220 kW.

It is evident from the graph that the worst conditions for generating peak capacity occur during the period of tidal amplitudes at quadrature. It is also evident that the generation of guaranteed peak capacity is possible only by pumping operations. /73/

The requirement of mandatory participation of the TPP in covering the peak of the load curve results in a restructuring of the entire TPP regime. For a long time the TPP operates almost as a one-way installation. From the 4th through the 8th days the TPP operates as a strictly one-way installation. From the 9th through the 11th days the output of the TPP is produced mainly by reverse turbine regime, while from the 12th through the 17th days it is produced by direct turbine regime.

Limiting the maximum TPP head under turbine regimes to 2.5 meters results in having to introduce waiting periods during the 13th day when the TPP head exceeds 2.5 meters. This causes a certain decrease in output which could be avoided by increasing the maximum head to 3 meters.

Idle discharges through the surface and bottom orifices play an important role during TPP operation under peak regime because they make it possible, by switching over to one-way operation, to generate the required peak capacity during syzygial tides (12th and 13th days). In addition, idle discharges are important during the period of switchover of peak capacity generation regimes from direct turbine to reverse (8th and 9th days) and from reverse to direct (11th day).

The insignificant capacity of the TPP in comparison with the capacity of the power grid naturally excludes the need for the TPP to operate in peak regime and limits this regime only to the needs of the experiment.

The total energy which the TPP can produce in one year, according to these calculations, with one turbine unit will be 1.2 million kW-hours and 2.3 million kW-hours with two units. Transition to peak operating regime results in a decrease in TPP output by approximately 20%.

The role of various TPP regimes is shown in Table 4-1. The TPP output for each variant of regulation is determined over two days with an average tidal amplitude. The table data and analysis of the regimes show the small effect of pumping operations on increasing output (2.3% for two-way operation and 0.6% for one-way operation), which confirms that it must only be used to ensure peak TPP regimes. The effect of the water passages turns out to be significantly larger (15.3% during two-way operation and 11.4% during one-way operation).

/74/

It should be noted that the cited data on the results of computer calculation of optimum TPP regimes by the method of differential dynamic programming are of fundamental significance in proving the importance of using stringent methods of calculation for regulating the TPP, inasmuch as they make it possible to increase the output of the TPP by approximately 30% in comparison with approximate methods.

The actual characteristics differ somewhat from the data used in the program calculations. Thus, the actual characteristics of the unit (Fig. 4-7) differ somewhat from the manufacturers characteristics (Fig. 4-6); the curve of changes in the basin area as a function of the water level (Fig. 2-6) is also not straight, as assumed in the calculation. For this reason the TPP regimes obtained by calculation are subject to refinement when factual data are introduced into the calculation. During actual management of the TPP regime under normal operation one should use a set of typical TPP control graphs for maximum output calculated by computer and setting the TPP operating regime at a particular instant in time t , depending on whether this instant

/75/

Table 4-1.

Role of various regimes in the operation of the Kislaya Guba TPP

Control variant	Regime	TPP output over 2 days		
		kWh	% two-way operation	% two-way operation under variant 4
1. Control taking into account pumping operations and the operation of water passages for idle discharge (surface and bottom)	T	9,114	100	117
	0	5,885	64	75
2. Control taking into account pumping operation but without taking into account water passages (surface bottom)	T	7,800	100	100.1
	0	5,501	70	70.5
3. Control without taking into account pumping operations but taking into account water passages (surface and bottom)	T	8,909	100	114
	0	5,852	66	75
4. Control without taking into account pumping operations or water passages for idle discharge (surface and bottom)	T	7,790	100	100
	0	5,274	67	67
5. Control taking into account pumping operations and operation of the bottom weir (without surface spillway)	T	8,052	100	103
	0	5,588	70	71.5
6. Control taking into account pumping operations and operation of the surface spillway (without bottom weir)	T	8,151	100	105
	0	5,650	69	72.5

Note: T - two-way operation, 0 - one-way operation.

t corresponds to ebb or flood, the actual water levels in the sea and the basin at instant t , the expected amplitude of the flood (ebb), and by how much instant t precedes the moment of high (or low) water.

At the present time, while the turbine unit is being tested at the TPP and its actual characteristics are being measured, the TPP is being operated on the basis of previously established manufacturers characteristics and on the basis of general recommendations obtained through computer calculations for optimum regimes at the Kislaya Guba TPP.

4-2. Hydroelectric power equipment

a) Hydraulic turbine unit

A bulb-type tidal reversible unit was chosen for the Kislaya Guba TPP because it ensures the most effective utilization of tidal energy under the conditions of a single-basin installation.

A hydroturbine of the adjustable blade type with a 3.3 meter diameter runner wheel and a rotational speed of 72 revolutions per minute is coupled through a planetary step-up gear (manufactured by "Krupp", FRG) with a synchronous generator of 400 kW capacity and 600 revolutions per minute. The step-up gear coupling turned out to be necessary due to the very low head at which the turbine operates (0.5-2.5 meters). The range of heads at the various TPP building sites along the White Sea coast may be increased to 1-7 meters because the tidal amplitude is significantly greater than in the Kislaya Guba (syzygial amplitude in the Kislaya Guba is 4 meters, in Lumbovka Bay it is 7 meters, in the Mezen it is up to 10.2 meters). Nevertheless, these amplitudes are lower than those observed at TPP sites planned in France and England (6-13 meters) and for this reason coupling the turbine through a step-up gear is more efficient along the Soviet coast. It is evident from the above why the Kislaya Guba unit is seen as a prototype of more powerful installations for future industrial TPPs.

The standard of Soviet hydroturbine construction made it possible to manufacture the horizontal axial-flow unit for the Kislaya Guba TPP. As early as 1963 [4] it had been proposed in the Soviet Union to use similar machines for the Kama hydroelectric power plant. However, the manufacture of a single unit for the small experimental Kislaya Guba installation was seen as inexpedient. The purchase of an imported installation for the Kislaya Guba TPP made it possible to take advantage of wide foreign (French) expertise acquired as the result of broad experience over 20 years of research, in which took part almost all of the West European turbine manufacturing firms which had built the hydroturbine units for the Rance TPP, for the creation of a Soviet bulb-type unit. As early as 1959, on the basis of study of a bulb-type unit made in France, L.B. Bernshtein proposed that it could be efficiently used at low-head river hydroelectric power plants [5]. Since then, at a number of river hydroelectric power plants (Kiev, Kanev, Cherepovets), 54 individual bulb-type units have been installed with significant improvement over the French prototype; the two units at the Saratov hydroelectric power plant are the largest ($D_1 = 7.5$ meters) and most powerful (45 megawatts) in the world. Under these circumstances the acquisition of the French unit for the Kislaya Guba TPP was all the more expedient in that it made it possible to concentrate the efforts of Soviet researchers on further improvement of the French bulb-type TPP unit as applicable to conditions in the USSR. The fact is, that, as mentioned above, the somewhat smaller tidal ranges at the possible Soviet TPP construction sites, than those in France, make it necessary to use a smaller head and to work within a wide range of heads (the head varies by 10-14 times). Under these conditions, a significant increase in efficiency, and consequently fuller utilization of the energy potential of the tide may make it possible to discontinue to maintain the number of revolutions of the turbine unit at a constant for direct operation and instead to switch over to

the variable operating speeds [4]. Special research carried out by the Moscow Power Engineering Institute and the All-Union Scientific Research Institute of Electric Power Engineering showed that this problem could be solved by using an asynchronized synchronous generator (ASG), developed by the All-Union Scientific Research Institute of Electric Power Engineering¹. At the present time, a system is also being developed to regulate the speed of the generator manufactured by the "Elektrosila" plant which is installed in the operating imported unit in place of the synchronous generator manufactured by "Al'stom". After full-scale tests of the operation of the unit with the ASG, final on-site determination will be made of the effectiveness of using variable speeds of rotation and, should it be necessary, such a unit will be manufactured and installed in the second turbine channel which is so far, as indicated above, being used as a bottom weir.

At the Kislaya Guba Tidal Power Plant [6] a horizontal bulb-type hydro-turbine has been installed which operates in six regimes (Fig. 4-3), with a cantilever configuration of the runner wheel 1, the load from which is taken up by the turbine bearing 2 and the thrust bearing 3. The moment from the turbine shaft is transmitted through an elastic "Takke" coupling 4 to the step-up gear 5 and then to the generator 6. The bulb 7 is protected from the possible penetration of sea water by graphite packing 8, which is installed on the runner wheel side.

The step-up gear is of the coaxial type, with a single step, three planetary gears, and a built-in uncoupling and recoupling mechanism. Lubrication and control of the uncoupling piece is accomplished by the pumps 9, located over the oil tank 11 in the fairing. The synchronous generator, with closed air cooling, is flexibly connected to the step-up gear and rests on two roller bearings.

¹Work supervised by M.M. Botvinnik, Doctor of Technical Sciences.

The fan is located on top and feeds hot air to the distributing coil 13 and then to the cooler, located in the fairing 10, around which flows the water. A special helical coupling connects the step-up gear with the runner wheel.

The runner wheel, with four adjustable blades, revolves in two bearings one of which is combined with the thrust bearing on the step-up gear side. The thrust bearing has a bushing on which the two discs of the swiveling segments are supported. Between them is located the oil header which feeds the servomotor of the runner wheel located in the hub fairing. The oil header is connected to the pipes of the control system by two flexible pipes, /79/ permitting it to follow the axial movements of the shaft. Seepage is carried to the fairing oil tank. The turbine bearing is protected by a deflector from the dampness of the graphite packing. The bearings and thrust bearings are lubricated by gravity feed 14 from the control system tank, which is regulated by a valve with an electric drive, located within the bulb. The rotor is equipped with a disc brake 15, consisting of two pneumatic jacks. The generator, step-up gear, brake, and thrust bearing are attached to the frame which rests on the ribs of the stator 16 and the two columns 12. The stator has three hollow ribs 17, through which pass the air ducts and pipes for removal of seepage.

The guiding mechanism consists of twenty vanes 18, operated by a control ring 19 driven by two servomotors. The vanes are connected to the ring by means of a connecting link 20 and a lever 21. The vanes are kept from jamming with the aid of a shear bolt 22. The servo motors have locks with oil servo motors for locking the vanes in the closed position. Integrated control is maintained during both direct turbine and reverse pumping operation. In the other regimes the vanes of the guide mechanism are locked in the open position.

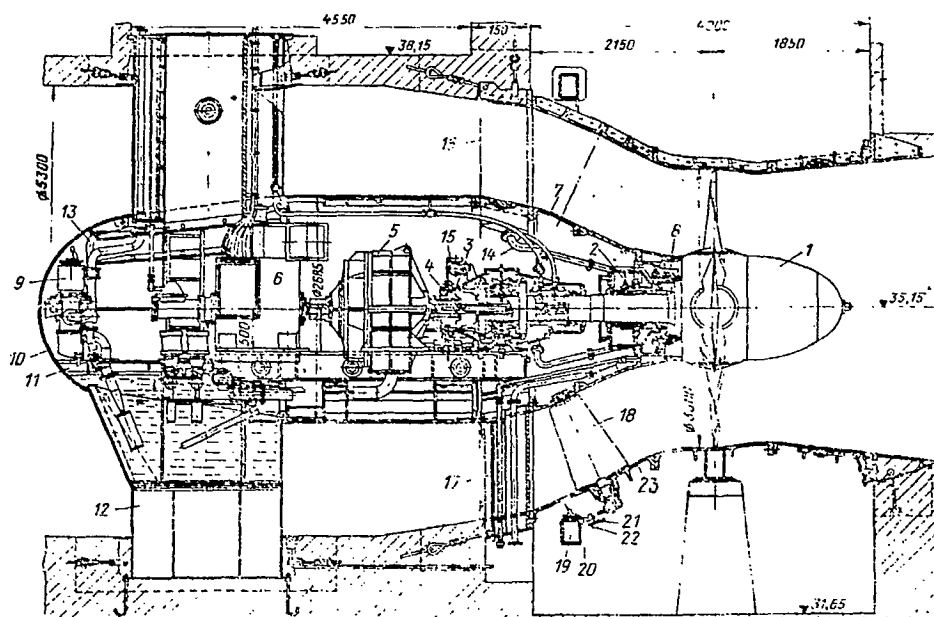


Рис. 4-3. Агрегат Кислогубской ПЭС.

1 — рабочее колесо турбины; 2 — подшипник турбины; 3 — подшипник комбинированный с подпятником; 4 — эластичная муфта "Такке"; 5 — мультипликатор; 6 — генератор; 7 — капсула; 8 — уплотнение; 9 — масляный насосный агрегат; 10 — верхний обтекатель; 11 — маслобак; 12 — опорная колонна капсулы; 13 — подача горячего воздуха в верхний обтекатель; 14 — маслопроводы для смазки подшипников и подпятника; 15 — дисковый тормоз; 16 — ребра статора агрегата; 17 — одно из трех полых ребер статора, через которые проходят воздухопроводы и трубопроводы дренажа капсулы; 18 — лопатка направляющего аппарата турбины; 19 — регулирующее кольцо; 20 — серьга; 21 — рычаг; 22 — срезной болт; 23 — конический обод (конусное кольцо) наружного водопаса; 24 — крошительны со стяжками.

Figure 4-3. The Kislaya Guba TPP hydroturbine unit.

1 - turbine runner wheel; 2 - turbine bearing; 3 - bearing combined with thrust bearing; 4 - "Takke" elastic coupling; 5 - step-up gear; 6 - generator; 7 - bulb; 8 - graphite packing; 9 - oil pump assembly; 10 - face fairing; 11 - oil tank; 12 - bulb support column; 13 - hot air supply to face fairing; 14 - oil supply lines for lubrication of bearings and thrust bearing; 15 - disc brake; 16 - ribs of the stator assembly; 17 - one of the three hollow stator ribs, through which pass the air ducts and the bulb drainage pipes; 18 - turbine guide vane; 19 - control ring; 20 - connecting link; 21 - lever; 22 - shear bolt; 23 - conical rim (conical ring) of the external water passage; 24 - supports with tightening devices.

The runner wheel chamber is supported by means of resilient supports, which compensate for deformation of the water passage, on two reinforced concrete blocks and is fastened to the conical rim (conical ring) of the external water passage 23, which is made up of two halves. On its lower flange an adjustable seal is installed which slides freely along the conical ring when the water passage expands. Twelve supports with tightening devices 24 maintain the circular shape of the conical ring around the seal.

The turbine unit is controlled by means of a start-stop device which distributes oil under pressures of up to 40 kgf per cm² to the servomotors of the guiding mechanism and the runner wheel. The turbine unit has separate systems for lubricating the bearing assemblies under normal operation and under the start-stop regime. Oil is supplied to the control system by means of a pressurized oil installation equipped with continuously operating pumps.

The special features of the hydroturbine unit indicated above caused certain difficulties during installation. These involved, first of all, special demands made on the shaft line and the support nodes. Lack of much experience in the installation of horizontal units (the Cherepovets and Kiev hydroelectric power plants had been built in the USSR by the time installation was begun), and the special features of many of the assemblies, complicated the assembly operations. In addition, the nature of some of the assemblies, such as the graphite packing, flexible coupling, etc, which are not typical of Soviet hydraulic turbine construction, must be taken into account.

/80/

Assembly was carried out by a five-man team from the Specialized Trust for Hydraulic Power Plant Engineering. Technical supervision was carried out by representatives of the Leningrad Metal Plant. The large parts were installed with the aid of a tower crane. The high speed of the creane hook caused difficulties in achieving the precision required during the joining of the assemblies and parts.

An assembly platform was constructed for inspection and preliminary assembly. The sequence of assembly was worked out by the manufacturer and presented in the form of a plan. For some of the operations which are typical during the assembly of Soviet hydroturbines, however, the manufacturer did not provide instructions.

Assembly of the turbine unit, which arrived in the USSR in February 1964, was only begun in 1966, for which reason its preliminary assembly was carried out on the assembly platform. Blocks which are not subject to inspection under normal delivery conditions were opened and also subjected to internal inspection.

Assembly of fitted pieces was carried out in accordance with the plan provided by the manufacturing plant. As each succeeding assembly was installed it was checked for coaxial alignment and verticality of the flanges. Special care was taken with the stator since it is the base assembly for installation of the guiding apparatus, the bearing supports, the runner wheel chamber and the suction pipe cone.

After installation of the supporting frame for the mechanisms within the bulb with the aid of a special hoist supplied by "Neyrpic", the generator and the step-up gear were installed. The driven part of the cogged flexible coupling was first fitted on to the male end of the step-up gear shaft. The fitting was carried out on the assembly platform after the coupling was heated in an oil bath.

In order to control clearances in the bearing, thrust bearing, and oil collector, preliminary assembly of the shaft was carried out on the assembly platform. Clearances were controlled with the aid of gauges.

/81/

Installation of the shaft in the bulb was carefully controlled for coaxial alignment since the position of the shaft affects the installation of the runner wheel, chamber, and the suction pipe cone.

Assembly of the bearing assemblies was a difficult operation as a result of their complexity and inaccessibility. This especially has to do with installation of the graphite packing which, apparently, was supposed to have been assembled with the aid of a special device.

Prior to the connection of the runner wheel with the shaft it underwent examination during which its internal mechanisms were inspected and its tightness was tested at a pressure of 2.5 kgf per cm^2 . The operation of the runner wheel servomotor was also tested. It should be mentioned that the quality of the corrosion proofing and the condition of the packing of the runner wheel were good. The runner wheel was lowered by crane and joined to the shaft flange (Fig. 4-4). Tightening of the bolts was regulated by the angle of turn with a bolt pressure of $1,800\text{--}2,000 \text{ kgf per cm}^2$. Two variants were examined for final installation of the shaft. The first variant consisted in coaxial alignment of the bearing and thrust bearing (Fig. 4-5, a). Under these conditions the bearings operate under optimum conditions.

If the step-up gear shaft is centered to the turbine shaft in this position, part of the load of the weight of the runner wheel on the turbine shaft may be transmitted through the "Takke" coupling to the step-up gear. In order to avoid this a variant was chosen under which the bearings were arranged out of coaxial alignment (Fig. 4-5, b). Centering to the step-up gear shaft was carried out in this position and, consequently, transmission of force to the step-up gear bearing was eliminated. Gauges were used during centering to control the alignment of the flanges of the step-up gear and turbine shafts, as well as their coaxial alignment. Misalignments did not exceed 0.03 millimeters. The same precision was achieved during centering of the generator shaft to the step-up gear shaft.

During installation of the guiding apparatus difficulties showed up in joining the outer rings, caused by their complex shape and residual deformations. For this reason, the surfaces to be joined were scraped. In addition, it should be noted that the clearances between the friction surfaces of the speed wheel were inadequate, which caused quite considerable displacement

stresses on the guide apparatus. A warp in the speed ring also contributed to this. In order to decrease the frictional forces the surfaces were scraped, upon which the minimum pressure for shifting of the guiding apparatus decreased from 12 to 7 kgf per cm^2 . After the guiding apparatus was assembled the runner wheel chamber and the suction pipe casing were installed. The oil and water pipelines, the pressurized oil and compressor equipment, and the control system were installed after the installation of the main equipment. After completion of the assembly work all of the hydroturbine systems were tested according to a special program in compliance with the manufacturer's recommendations and Soviet standards.

/83/

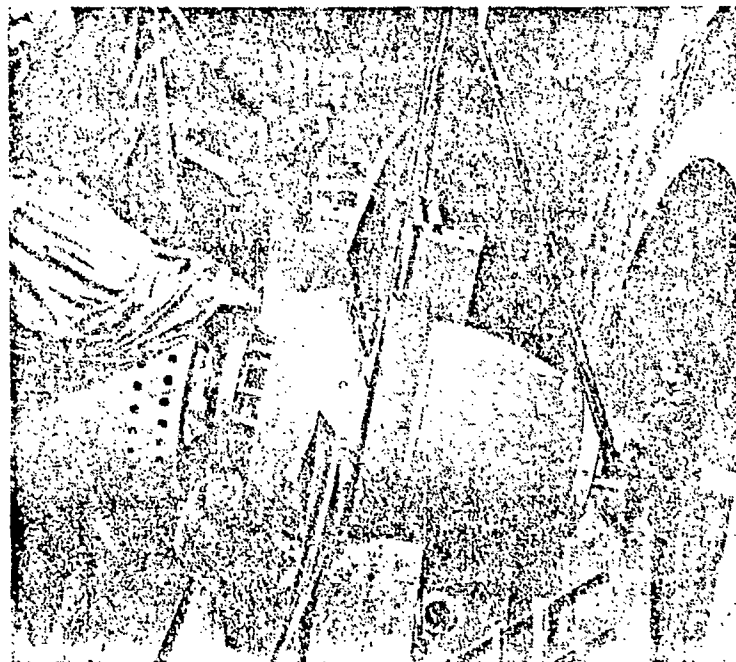


Рис. 4-4. Стыковка рабочего колеса с валом турбины. 11/VII 1966 г.
Фото А. С. Фирфарова.

Figure 4-4. Joining of the runner wheel to the turbine shaft. 11 July 1966.
Photographed by A.S. Firfarov.

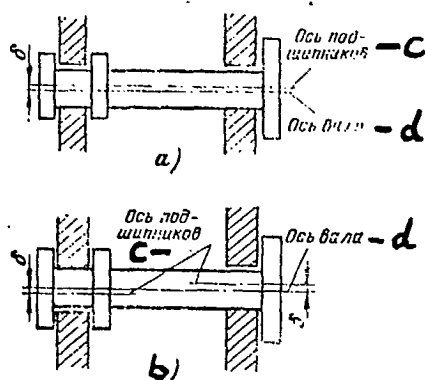


Рис. 4-5. Установка подшипников турбинного вала.

а — вариант соосного расположения подшипника; б — вариант несоосного расположения подшипника.

Figure 4-5. Installation of the turbine shaft bearings.

a - coaxial bearing alignment variant; b - non-coaxial bearing alignment variant; c - axis of bearings; d - shaft axis.

Testing in the construction dock was completed by motoring the unit in direct pumping regime.

The main characteristics of reliable operation of the assemblies (run-out of the shafts, vibration of the supports, temperature of the bearing and thrust bearing) did not exceed permissible limits.

Prior to start-up of the unit, after submergence of the TPP floating block at the site, the operation of the mechanisms and control systems was retested. The first start-up was carried out under manual control, in direct turbine regime, without inclusion in the grid. When 75% of the nominal revolutions were obtained, measurements were taken of the run-out of the turbine, generator, and step-up gear shafts, the bearing temperature, vibration, pressures, and the rotating and supporting assemblies were inspected and tested for noise in order to detect extraneous noises, catching, leaks, and other operational abnormalities. Thereafter the revolutions were increased to nominal speed and maintained by the vane turning mechanism until the temperature of the bearings and thrust bearing was stabilized. During acceleration

of the unit, in order to test the operation of the uncoupling collar, the unit was stopped and started up automatically, taking on and then dropping load.

Tests showed (Table 4-2), that during operation of the unit the amplitude of vibrations of the bearings and runner wheel chamber did not exceed 0.02 millimeters; the run-out of the shaft in the area of the turbine bearing, step-up gear and generator did not exceed 0.02-0.03 millimeters under all regimes. No leaks were found within the chamber. The unit operated smoothly under all regimes. Under reverse pumping regime a rotating frequency hum could be heard in the runner wheel chamber. The temperature of the bearings did not exceed 35°C.

/84/

The test results showed that it would be necessary to conduct additional special tests:

- 1) to establish the necessary time intervals for the servomotors of the guiding apparatus and the runner wheel, needed to ensure synchronization with the grid during start-up of the hydraulic turbine and rpm increase of not greater than 108% during shut-down from idle run regime under all heads. The times may be changed with the aid of choking devices and by changing the running speeds of the motors of the mechanisms for opening the guiding apparatus and the runner wheel;

- 2) to provide for the regulation of gravity lubrication of the hydroturbine bearings with the aid of, for example, a jet relay;

- 3) to investigate the transition process during drop of load or emergency stop with the aim of determining the optimum increase in revolutions and pressure in the flow-through part of the hydroturbine under various regimes;

Table 4-2

Results of running vibration tests of the Kislaya Guba TPP unit

Regime	Head, mm	Vane Opening, %	Opening of vanes as a fraction of full-open	Capacity, kW	Turbine bearing temperature, °C	Step-up gear bearing temperature, °C	Turbine shaft run-out, mm	Generator shaft run-out, mm	Shaft coupling run-out, mm	Vibration, mm	Chamber vibration, mm
Direct turbine	0.62	55	0.8	80	28	28	0.01	0.03	0.03	0.01	0.00
Direct pumping	0.43	80	1.0	400	34	29	0.01	0.02	0.02	0.01	0.01
Reverse turbine	0.72	59	1.0	80	1	26	0.01	0.02	0.02	0.03	0.00
Reverse pumping	0.48	2	1.0	400	26	26.5	0.1	0.01	0.01	0.01	0.01

4) to carry out measurements of the forces acting on the guiding apparatus and the runner wheel under all operating regimes.

Hydraulic power engineering research and testing of the unit under the blower regime were carried out by the Scientific Research Department of the All-Union Scientific Research Institute for Design and Exploration while it was still in the construction dock at Cape Prityka, before the unit and the foundation area were flooded. These tests were carried out in order to establish the reliability of the mountings of the unit and to see how it would normally operate with its unusual thin-walled design. Analyses of the vibrational characteristics of the main members of the unit showed that their vibrational load under the most difficult reverse pumping regime is 2-4 for the generator, 2-6 for the bulb, 5-20 for the thrust bearing and 7-24 nonometers for the runner wheel chamber. According to the data of the State Trust for the Organization and Rationalization of Regional Electric Power Plants and

/85/

Networks, for a unit with a runner wheel diameter of this size, the vibrational status is considered to be excellent if the amplitude of vibrations of the elements does not exceed 45 nanometers. The measurements which were carried out showed that in the Kislaya Guba machine only the step-up gear has a vibration range of 55 nanometers. But considering that the measurement of these vibrations was carried out on the cover of the casing, the actual vibration of the shaft axis will also give a figure of 40-45 nanometers. Testing of the unit at the site, under operating regime conditions, showed that most of the manufacturers guarantees were satisfied (under direct turbine regime, at the rated head of 1.28 meters, the capacity exceeded the guaranteed 400 kW and reached 410 kW), with the exclusion of the starting head under direct turbine regime which ensures stable operation (the head was found to equal 0.8 meters instead of the 0.18 meters guaranteed by the manufacturer). In addition, once the racing control system was run in, direct idle throughput became possible at a head of 0.8 meters instead of 1.28 meters and reverse throughput became possible at 1.0 meters instead of 1.62 meters (Figs. 4-6 and 4-7).

Further studies made it possible to plot a combinatorial curve which ensures optimum operation of the unit (Fig. 4-7) and which, as usual, differs significantly from the factual curve provided by the manufacturer (Fig. 4-6). Transitional regimes, the effect of swell, and factual characteristics of the unit will be studied in the future.

The hydroelectric power equipment, including the control system equipment, and its layout in the confined thin-walled block structure, were designed jointly by the French firms "Neyrpic" and "Al'stom," together with the Soviet All-Union Planning, Surveying and Scientific Research Institute.

b) Auxiliary Equipment

The auxiliary systems, which are standard at every hydroelectric power plant, consist of the following at the Kislaya Guba TPP.

The pneumatic system includes two compressors, working and reserve, of the "Lyushar" type (delivery 0.2 m^3 per minute, pressure- 35 kgf per cm^2), which provide for the operating requirements of the unit: the hydraulic control system- 35 kgf per cm^2 , and the hydraulic brake system 12-kgf per cm^2 .

/87/

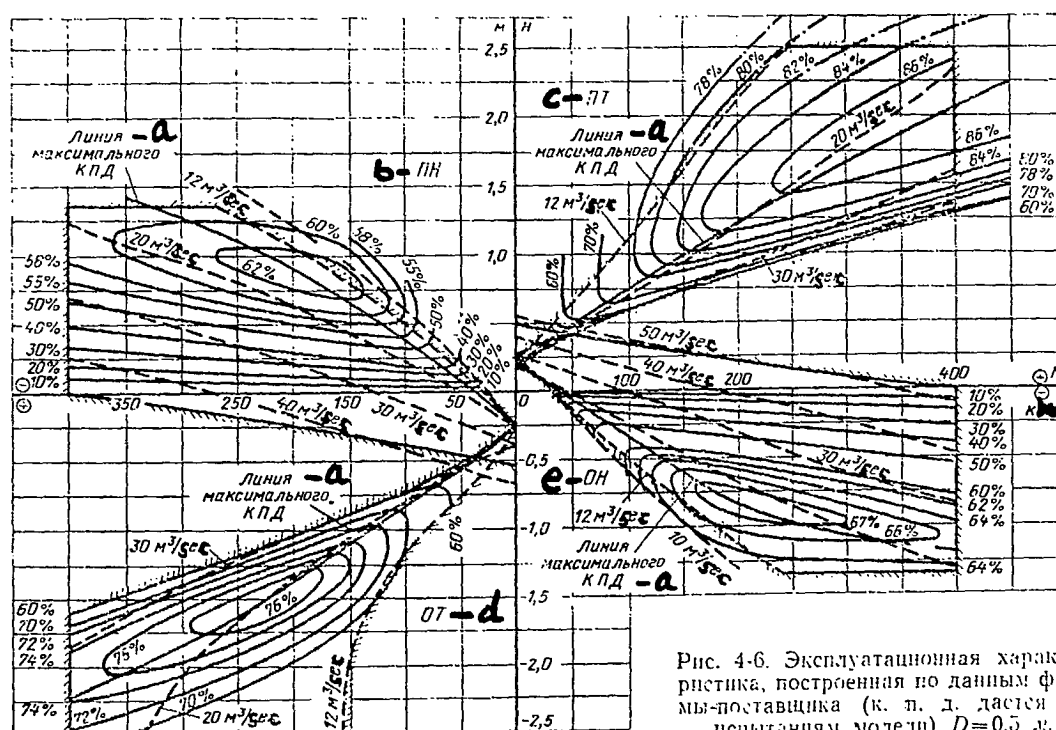


Рис. 4-6. Эксплуатационная характеристика, построенная по данным фирмы-поставщика (к. п. д. даны по испытаниям модели) $D=0,5 \text{ м}$.

Figure 4-6. Operational curve plotted from the manufacturer's data (efficiency given from model tests) $D = 0.5 \text{ meters}$.

a - maximum efficiency curve; b - direct pumping; c - direct turbine; d - reverse turbine; e - reverse pumping.

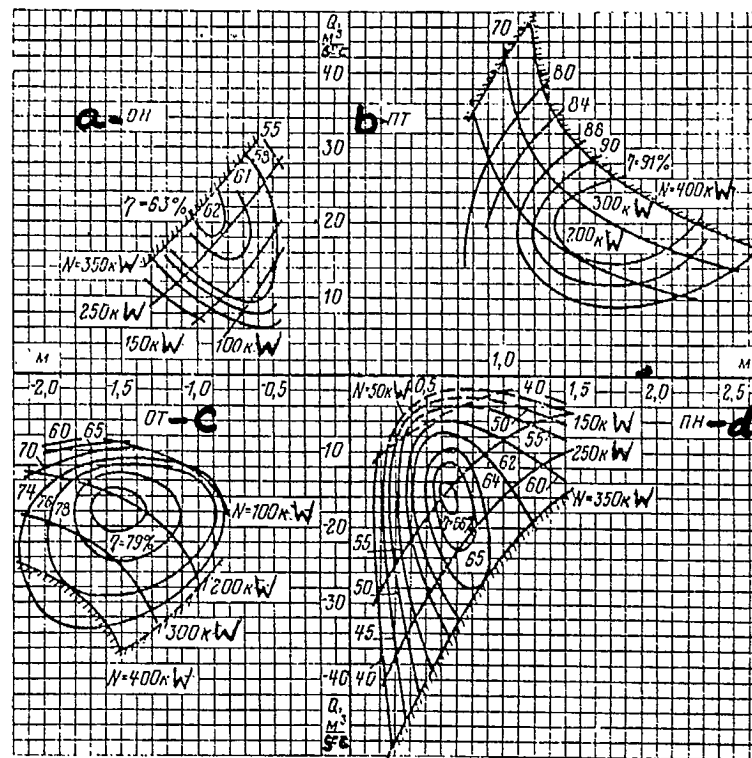


Рис. 4-7. Эксплуатационная характеристика, построенная на основании данных натурных испытаний ОГЭИ НИС Гидропроект, с нанесением линий ограничения по углам разворота лопастей рабочего колеса в зависимости от напора при работе агрегата в зоне максимальных мощностей.

Figure 4-7. Operational curve, plotted on the basis of full-scale test data compiled by the Chief Power Engineer's Section of the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, with the limiting lines plotted for the setting angles of the vanes of the runner wheel as a function of the head during operation of the unit in the zone of maximum capacities.

a - reverse pumping; b - direct turbine; c - reverse turbine; d - direct pumping

In order to supply the pneumatic instruments and other equipment, a VU-3/8V compressor was installed (delivery- 3 m^3 per minute, pressure- 8 kgf per cm^2).

It must be noted that special attention has to be devoted to installation of compressor equipment under such thin-wall design conditions. Thus, for example, installation of the VU-3/8V compressor on a special 3.0 m^3 reinforced

concrete base completely eliminated the danger of resonance vibrations in the adjacent structures but turned out to be insufficiently effective for damping audible frequency vibrations. In the future, compressors should, it appears, be installed on special shock absorbers.

The oil system includes four reinforced concrete oil tanks, faced with metal, which were built into the block structure, a control room with a centrifuge and a filter press, an oil line system with a capacity of 3 m³ per hour, and tanks for emergency drainage. The presence at the TPP of the complete set of oil supply equipment is due to the TPP's remoteness and difficult accessibility. The construction of the emergency drainage tanks is most interesting. The TPP structure, which is constructed directly beneath the turbine chamber (Fig. 3-2, a) is in the form of separate reinforced concrete spaces with a double-layered epoxy lining. This is the first attempt in the history of Soviet hydraulic engineering to use reinforced concrete oil tanks in which the metal lining is replaced with a protective epoxy coating. This design turned out to be reliable, efficient, and cost effective.

After examination of a series of variants, the pumping and drainage system was classically designed, incorporating pump equipment new to hydraulic power plants.

The water from the drainage system of the TPP structure and the seepage from the turbine chamber run down to the lower levels of the structure and accumulate in two chambers similar to the tanks for emergency discharge of oil. The water is pumped out of these chambers by vertical centrifugal pumps of the NTsV25/30 types (delivery-25 m³ per hour, head-30 meters water column). Operation of the pumps is automated from an electrode relay at the level of the new structure which, in contrast to the electronic level indicator relay type, has a smooth response level. The construction of the sensing device ensures its reliable operation under fresh and salt water conditions and in the event of accidental leakage of oil into the drainage system.

Due to the relatively large volume, pumping-out of the flow-through part of the turbine could only be accomplished by using a water gallery, to which was assigned the space adjacent to the turbine chamber. Vertical pumps of the NTsV250/30 type (delivery-250 m³ per hour; head-30 meters water column) are located on special tubular supports on the wall of the turbine chamber on the sea side. Operation of the pumps is automated by means of an electric contact vacuum manometer, installed in the water gallery. The use of vertical pumps of the NTsV type, with installation both directly on the deck and the wall of the turbine chamber, made it possible to solve the difficult problem of placing two pumps in very restricted quarters. Intensive operation of the pumps, especially during initial operation of the experimental block, demonstrated their high reliability and ease of servicing (Figs. 3-2 to 3-4).

4-3. Mechanical Equipment¹

The TPP equipment includes emergency and maintenance gates for the hydroturbine unit and the bottom weir, maintenance gates for the spillway, protective screens, and cranes.

Gates. The water passages of the hydroturbine unit and the bottom weir may be closed off by flat deep-seated sliding single-plated gates (5.76 x 5.65 x 11.23 meters). The gates are designed to be lowered into flowing water and to withstand pressure from both sides.

The surface spillway may be closed off by flat sliding gates 5.76 x 6.00 x 3.2 meters. The spillway gates are installed on cross supports in the gate guides of the deep-seated gates of the turbine's water conduit. This combination of the gate guides was made possible by the fact that the surface spillway was, on the whole, excluded from the operating cycle of the TPP and was intended only for research work. The presence of even a single

¹Mechanical equipment of the TPP was jointly designed by the All-Union Planning, Surveying and Scientific Research Institute and the Special Design Office of Mosgidrostat.

gate on the surface spillway makes it possible to create a head on the TPP and does not interfere with closing of the turbine gate in the event of an emergency shut-down.

Protective screens. In order to protect the turbine from the penetration of floating objects, seaweed and seals, capron and metal nets with a mesh of 120 x 120 millimeters, stretched over a metal framework, were installed in the gate guides of the deep-seated gates. At low hydraulic resistance they provide reliable protection of the turbine unit.

/90/

Hoisting equipment. The presence of much mechanized equipment of various types, the restricted dimensions of the TPP block, and the need to carry out other operations made it necessary to create a universal hoisting apparatus. We will list only the main operations which must be carried out by this equipment:

manoeuvring of the deep-seated gates of the spillway and the turbine unit from both the sea and basin sides;

manoeuvring of the surface spillway gates;

assembly and disassembly of the units, and also maintenance operations on the upper deck of the TPP building and the spillway and through the covers in the blocks of the turbine unit and the bottom weir;

installation of the gates in the gate housing, located in the west dam;

delivery of small loads by a 5 ton-force capacity hook to the TPP structure through the freight shaft;

unloading of floating equipment moored to the TPP block onto trucks;

installation of protective screens.

During construction of the hoisting equipment it was also necessary to take into account, firstly, that the upper deck of the TPP structure is used as an assembly platform and for this reason its usable dimensions should not be significantly decreased, secondly, the strong (up to 50 meters per second) winds made it necessary to design a structure with the lowest wind resistance, and thirdly, this apparatus, which crowns the TPP structure, must give it its architectural style.

The most expedient of the many variants examined turned out to be a fully rotary-jib gantry crane.

The crane moves on tracks along the surface spillway, has a gantry height of 5 meters and a jib with two hooks: 30 ton-force at an overhang of 11 meters and 5 ton-force at an overhang of 13 meters. A special feature of the crane is its load-bearing jib, which forms a single unit with the entire rotating assembly, including the built-in motor area and the control cabin. This made it possible to design the metal construction of the rotating part more efficiently, to give the crane a streamlined appearance, and to significantly improve its architectural style (Fig. 3-3).

/91/

Hydraulic hoist. The bottom weir takes part in the operational cycle of the plant and for this reason it must be operated up to 8 times per day. In order to maneuver these gates a vertical hydraulic hoist was installed with a hoisting force of 40 ton-force, a lowering force of 5 ton-force, and a rod length of 6 meters. The gate control and the gate position indicator were brought out to the plant control panel.

Anti-icing protection of the gate guides. In order to prevent icing of the gate guides of the constantly operating gate, together with the hydraulic drive, there was constructed a thermally insulated wall along the pier caps from the intake wall to the upper deck of the TPP structure, in order to create a space protected from the atmosphere and heated by the exhaust ventilation from the TPP structure.

In compliance with the recommendations of the All-Union Scientific Research Institute of Hydraulic Engineering, the gate guides of the occasionally used spillway gates were equipped with a system of static oil heating, accomplished by means of six heating pipes located within the reinforced concrete near each gate guide along its entire length. The pipes are filled with transformer oil, and electrodes, consisting of steel rods 6 millimeters and 10 millimeters in diameter, wired in series, are lowered into them.

4-4. The electrical system

The Kislaya Guba TPP is tied in to the Kola power grid by an electric transmission line 16.4 kilometers long and carrying 35 kilovolts. The line traverses very rugged terrain which rises to a plateau located on high precipitous cliffs near Ura Guba and which is split by deep ravines through which flow streams full of rapids and on which there are depressions containing shallow lakes. Due to the difficult conditions and absence of roads and approaches to the path of the transmission line, almost all of the cargo and personnel were delivered to the line by helicopters, all-terrain vehicles, and by boats from the sea. The towers were built of wood with an original system of supports consisting of tension members anchored in the rock. The electric transmission line approaches Kislaya Guba from the direction of the high west bank and, after spanning the inlet, it leads to the outside distribution system which is located on the east bank (Fig. 3-1). At the outside distribution system there are two 35/0.4 kilovolt step-down transformers: one transformer with a capacity of 560 kilovolts-amperes with control of voltage under load, another transformer with a capacity of 630 kilovolt-amperes without control of voltage under load, which excludes the possibility of their operation in parallel but ensures power from the second transformer for local use (during pumping regime of the turbine unit). The use of a

transformer with control under load was determined by the operation of the plant in the generating and pumping regimes. A building is located next to the outside distribution system (Fig. 3-1) in which are installed four 0.4 kilovolt power panels and plant power requirement assemblies (Fig. 4-8).

In the event of break-down of the 35 kilovolt transmission line a complete diesel-electric power plant with a capacity of 100 kW was built to ensure the delivery of electric power to the most important users of the TPP.

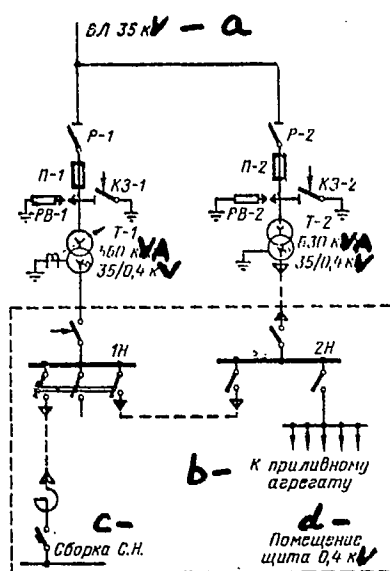


Рис. 4-8. Схема подстанции 35/0,4 кВ Кислогубской ПЭС.

Figure 4-8. Circuit of the 35/0.4 kilovolt substation of the Kislaya Guba TPP.

a - overhead line 35 kilovolts; b - to turbine unit; c - medium voltage assembly; d - control panel building 0.4 kilovolt.

The 35/0.4 kilovolt transformers operate with dead-earth neutral wire. There is a common grounding installation for the TPP and outside distribution system. Because the TPP is located in a region in which the earth has a high specific resistivity, the grounding installation is in the form of remote grounds (the metal sheet pile walls of the connecting dams). The casings of five aluminum power cables, with a cross section of $3 \times 120 \text{ mm}^2$, layed between the TPP and the outside distribution system in a special channel, are used to connect the grounding installations of the TPP and the outside distribution system. There is also a special cable for this purpose.

The outside distribution system is protected from commutational overloads by means of a 35 kilovolt circuit breaker installed immediately next to the transformers. Inasmuch as the main 35 kilovolt electric equipment of the outside distribution system is situated beneath the line there is no provision for protection from direct strikes by lightning.

/93/

Cable connections with the equipment located in the bulb unit (generator, transformers, oil pumps and step-up gear coupling collar, drainage pump, generator fan, pressure and temperature gauges, etc), were installed through the inspection shaft in the upper support of the bulb unit.

The automatic controls are designed in such a way as to enable the unit to operate under six regimes, depending on the direction of flow.

Direction of flow basin-sea:

Direct turbine (DT),

Reverse pumping (RP),

Direct throughput (DTP);

Direction of flow sea-basin:

Reverse turbine (RT),

Direct pumpinp (DP),

Reverse throughput (RTP).

Control of start-up and shut-down of the unit is executed either automatically or manually. Transition from one regime to another may be carried out manually only in a definite sequence: DT - DTP - RP - RT - RTP - DP - DT, etc and the transitions DT - DTP - RP and RT - RTP - DP (i.e. when the flow is in one direction) may be carried out without stopping the unit.

For example, during ebb the unit operates under direct turbine regime (direction of flow from the guiding apparatus to the vanes). When the head falls below 0.5 meters the unit is switched to direct throughput regime: it is switched out of the grid, the guiding apparatus and the vanes of the runner wheel are locked in the fully open position.

In order to accelerate equalization of levels in the basin and the sea and, if it is necessary (for optimum operating conditions), after equalization the turbine unit is switched back into the grid in the reverse pumping regime. The water is pumped out of the basin until, as a result of the tidal flow from the sea side, the head attains 0.5 meters; the turbine unit is switched on in the reverse turbine regime, etc.

In the DT, RP, and DTP regimes the unit rotates counterclockwise (when observed from the generator side), in the RT, DP, and RTP regimes the unit rotates in the clockwise direction. When the direction of rotation changes the contactor on the generator leads changes its phasing sequence.

/94/

The control system of the unit has a normal pressure of 38 kgf per cm^2 . The pressure is regulated in the forced-oil system tank with the aid of two pumps (working and reverse) operating with distributor-and-contact breaker units. The distributor-and-contact breaker unit, which maintains the pressure in the forced-oil system tank between the limits of 38-35 kgf per cm^2 , makes it possible to reduce the expenditure of energy on the constantly operating pump by switching it, once the 38 kgf per cm^2 pressure is attained, to a closed operating cycle regime with a pressure of 4 kgf per cm^2 , necessary to lubricate the pump.

In addition to charging the forced-oil system tank, the compressor installation is also used to maintain a pressure of 10 kgf per cm² in the braking receiver. In order to lubricate the turbine bearing during start-up and shut-down there are start-up pumps which create a pressure of 200 kgf per cm². Normal lubrication of the bearing and thrust bearing of the turbine is accomplished by gravity feed from the control tank.

The turbine unit is regulated by means of the vane control device and the servo system which controls the opening of the guiding apparatus. The extent to which the guiding apparatus is opened in the direct turbine and reverse pumping regimes depends on the head and the angle of pitch of the vanes of the runner wheel. In idle throughput, reverse turbine, and direct pumping regimes the guiding apparatus is locked in the fully open position.

The differential pressure gauge consists of two manometric boxes which sense the pressure of the water from the sea and basin sides and of a force-balancing bridge through which passes current proportional to the difference between these pressures. The current from the gauge passes as input to the electronic equipment, which changes the incoming signal into a command pulse which causes one of two polarized relays to trip, which in turn switches on an auxiliary relay controlling the electric motor which moves the combinatorial wedge.

/95/

The unit does not have a speed governor and for this reason cannot operate under the generator regime without being tied in to the system.

When started up in the turbine regime the unit accelerates to 90% of nominal speed and switches into the network. Excitation is delivered with the rotor slippage control after the main generator contactor is switched on.

The tachometer generator, which is connected to the turbine shaft by a friction drive, generates a voltage which goes to a voltmeter-rpm indicator and relay, which control the start up pumps, switching of the main generator contactor, and the braking system.

The electronic equipment supplied by the "Al'stom" Company includes: three automatic relay panels, a relay protection panel, a generator contactors panel, a power excitation panel, an excitation control panel, two control and signalling panels, two panels for power contactors and circuit breakers for the unit's power requirements, a panel for the charging and recharging rectifier equipment, and a storage battery.

The 60 ampere hour storage battery consists of 80 cadmium-nickel elements and operates in a buffer regime with the rectifier equipment, which maintains a tension of 110 volts on the bus bars.

The parameters of the rectifier equipment are as follows: nominal power supply voltage (3-phase)-380 volts, nominal rectified voltage-120 volts, nominal rectified current 15 amperes.

The rectifier equipment ensures automatic maintenance of the voltage at the required level under the "recharging" regime and stabilization of the current under the "charging" regime. Provision has been made for automatic switching of the rectifier equipment from the "charging" regime to the "recharging" regime when the voltage at the battery terminals attains 145 volts.

Of special interest is the excitation system for the generator, using semiconductor controlled rectifiers (thyristors) (Fig. 4-9). The excitation system consists of an excitation power transformer EPT, a regulating block RB, a control block CB, and a power rectifier block PRB.

The blocks are installed on a separate panel equipped with a fan for forced air cooling of the power rectifiers and equipment. The transformer EPT is wired to the generator bus bars. Current can be delivered to the

transformer only after a generator contactor K-1 or K-2 is switched on simultaneously with the delivery of current to the stator windings. Thus, the turbine unit can be switched into the network both by self-synchronization under the turbine regime and by asynchronous start-up under the pumping regime.

The regulating block RB has a metering part - a voltage adjuster with a manually operated voltage controller rheostat R_{cont} , a unit for limiting the rotor current, a unit for compounding the corrector, an intermediate full-wave two-stage amplifier with four transistors, and a two-transistor output power amplifier. The regulating block produces a voltage within the limits of +12 to -12 volts, which is fed to the control block CB.

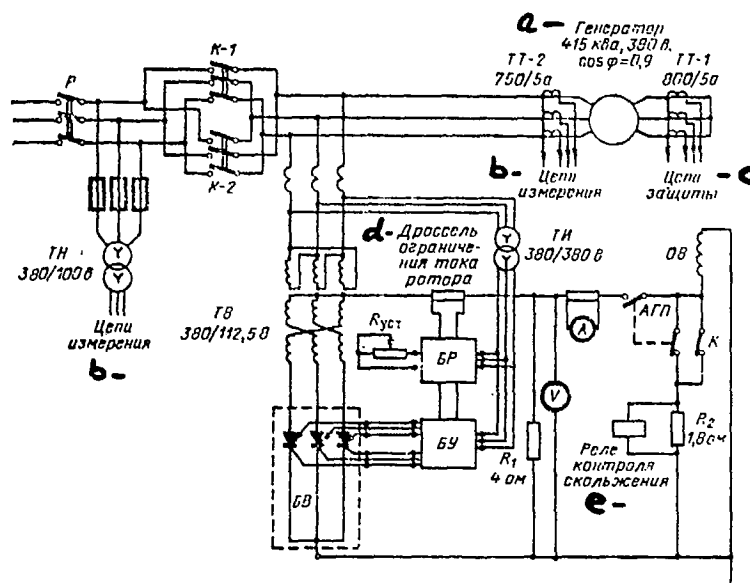


Рис. 4-9. Схема возбуждения генератора Кислогубской ПЭС на управляемых транзисторах.

Figure 4-9. Excitation circuit diagram of the Kislaya Guba TPP generator on controlled transistors.

a - generator, 415 kVA, 380 V; b - metering circuits; c - protection circuits; d - rotor current choke coil; e - slippage control relay; TT - current transformer; TH - voltage transformer; TB - excitation power transformer; TN - instrument transformer; $R_{уст}$ - voltage controller rheostat; АГП - automatic field damper; ОВ - field winding; БР - regulating block; БВ - control block; БД - power rectifiers block.

In order to add redundancy to the regulating block, to be able to assess the performance and to tune individual components of the block during adjustment, a redundancy feature was built into the regulating block. This unit consisted of a transformer, a full-wave rectifier bridge and a regimes switch for switching the excitation circuit from automatic regulation by the regulating block to manual regulation from the redundancy unit. In both cases excitation is varied by the rheostat R_{cont} .

The control block CB produces pulses to control the power rectifiers. The block circuit consists of a transistorized two-stage amplifier and a transistorized pulse generator. The phase of the pulse which triggers the power rectifier, and the interval of time for which the rectifier is open depend on the voltage supplied from the control block. In this manner the mean value of the rectified current varies. The control pulse remains constant in magnitude.

A special relay with nine three-way mercury contacts is used during switching of the reversible contactors of the turbine unit in order to match the voltage phases fed to the power rectifiers and the "network" voltages supplied by the regulating block RB.

The control and regulating blocks require 0.2 amperes per phase. The power rectifiers block PRB consists of three silicon thyristors incorporated in the phases of the secondary transformer winding EPT having a transformer ratio of 380/112.5 volts.

The transistorized automatic excitation control system AEC has a fast response time and is very durable. There are no facilities for overloading or de-excitation.

It should be mentioned that the "Al'stom" Company did not provide circuit diagrams for the internal connections of the regulating and control blocks or a description of the operation of the excitation circuit. Catalog

data and characteristics of the transistors were also missing, which made it more difficult to recalculate the circuits for operation with Soviet semiconductor devices¹.

¹All installation of the electrical equipment of the TPP (imported and Soviet) and start-up and adjustment operations were carried out by the State All-Union Electric Construction and Installation Trust.

Chapter 5

STATISTICAL CALCULATIONS OF THE FLOATING TPP STRUCTURE
AND FULL-SCALE STUDY OF ITS CONDITION UNDER STRESS5-1. Initial Situation

The turbine unit block of the Kislaya Guba TPP is a thin-walled reinforced concrete box-like structure composed of longitudinal and transverse flat members which support two horizontal reinforced concrete water conduits (Figs. 3-2 to 3-4).

/98/

The main supporting members of the structure are the walls of the end and middle buttresses, the longitudinal and transverse core-walls, the foundation plate, the walls of the suction pipes, and the horizontal deckings.

The external stresses acting on the block as a whole or on its individual members include: the weight of the reinforced concrete framework itself, the hydrostatic pressure of the water (alternating), the wave pressure, the weight and lateral pressure of the ballast fill in the empty spaces of the structure, pressure from the side of the rock-fill connecting dams, the suspending and percolating pressure of water at the bottom, the effects of temperature, loads from the equipment, dynamic loads from the flow in the turbine water conduit, the bottom weir and the surface spillway, and the operation of the hydroelectric power equipment.

Specific loads characteristic of the floating method of construction are loads arising during towage of the block.

Design of the thin-walled three-dimensional construction of the Kislaya Guba TPP block presents significant difficulties. Bearing in mind the significance of carrying out such construction in order to utilize the experience of its construction and design in the future, for powerful TPPs and low-head river hydroelectric power plants, we set up a program of

theoretical and experimental research on the condition of the block under stress which was included in the plan of the most important scientific research operations of the coordinated plan of the State Committee of the Soviet of Ministers of the USSR for Science and Technology and carried out by the All-Union Planning, Surveying and Scientific Research Institute with the participation of a number of specialized organizations: State Planning, /99/ Design and Scientific Research Institute of Marine Transportation of the Ministry of the Maritime Fleet, USSR, All-Union Scientific Research Institute of Transportation Construction, Moscow Construction Engineering Institute, Central Scientific Research Institute of Structural Parts, and the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute.

It is obvious that to apply elementary computational methods of the strength of materials to such an irregular cellular structure could give only very approximate results.

Indeed, a block framework composed of members of varying strength and at different distances from each other creates a real structural anisotropy. The ratio of the box's length to its height is approximately 2.3 which, in terms of the nature of the work of the block along the flow, corresponds to beam-walls designed in accordance with formulas of the theory of elasticity. Even this latter scheme, however, does not correspond to reality due to the significant width of the block, as a result of which assumptions about the equal distribution of normal and shear stresses along the face of the transverse section do not correspond to the true picture.

An additional difficulty in the design of this structure was the fact that the magnitudes of a number of external loads could not be included in the calculations with the proper degree of precision either due to their indefinite nature (for example, the pressure of the ballast fill in the internal cells of the block, certain dynamic loads), or because of insufficiently reliable theoretical magnitude and their combinations (for example, for temperature and wave effects, the deformation modulus of the foundation).

Due to the fact that the precision of a calculation should not exceed the precision of the initial data, the main calculations of the stressed-deformed state of the TPP block, according to which the cross-sections of the main elements and their reinforcement were selected, were carried out using simplified design schemes with arbitrary differentiation of the stressed state of the block in general and locally, by independent calculations along and across the flow, by isolation of individual members without precise consideration of their interaction with neighboring members, etc. At the same time, theoretical research on the operation of this structure was carried out by more precise methods including the methods of the theory of elasticity. Bearing in mind the practical impossibility (due to the great complexity of the structure) of obtaining sufficiently reliable theoretical design data, experimental research was carried out.

The turbine unit block under discussion operates as a floating seagoing structure during the time it is towed and as a normal water engineering structure after it is installed at the site.

/100/

In view of the fact that the towing period is very short (less than 1 day), and the loads on it during this period, as calculations have shown, are significantly below operational loads, the turbine unit block may be examined as a water engineering structure of the normal type and it may be

designed according to the accepted norms and technical conditions for the design of concrete and reinforced concrete structures.

The turbine unit block of the Kislaya Guba experimental TPP is in the IVth class by strength in accordance with the Construction Norms and Regulations, II-I 1-62, for classification of water engineering structures. The rated quality of the concrete is 400. The steel for reinforcing the structure is of the 25G2C type ($\sigma_T = 4,500$ kgf per cm^2). The permissible width of gaps was specified by Construction Norms 55-59: a) for the underwater zone with pressure gradients greater than 20, the permissible gap width was less than or equal to 0.1 millimeters; b) for structures located in the variable level zone, the permissible gap was set at less than or equal to 0.05 millimeters.

In individual members of the structure, in the lower part of the side walls, the width of the gaps turned out to be somewhat larger than permissible (up to 0.16 millimeters).

Such deviations from the standards were allowed for the following reasons:

- 1) the presence of tar-epoxy resin waterproofing on the surfaces of the concrete which come in contact with water;
- 2) the presence of cathodic protection from corrosion of the reinforcing steel, which eliminates limitations of the size of open areas as a consequence of the aggressivity of water;
- 3) testing of full-scale parts of the structure, carried out by the Scientific Research Department of the All-Union Planning Surveying and Scientific Research Institute in 1964 showed that, in actuality, the gaps reach their calculated widths at loads exceeding those calculated theoretically.

5-2. Calculations of the Static Work of the Block Longitudinally

The longitudinal work of the block is made possible by sufficiently rigid longitudinal connections which either take up loads directly or take up loads by supporting reactions from the transverse members. Such connections include: the foundation plate, the walls and deckings of the turbine water conduits, the longitudinal bulkheads, the side plates, as well as the deckings in the upper levels of the structure.

/101/

During design and planning of the structure, calculations along the direction of flow were carried out by the strength of materials method in terms of girders with a complex cross section (equivalent girders) on an absolutely rigid foundation. All longitudinal connections within the cross-section under consideration were included in the equivalent girders cross-section, provided that their connections with other members ensures their complete participation in the flexing of the block. The four following computational cases are examined, including three maintenance cases and one operational:

a) computational case number 1 (maintenance). The water level in the basin is 44.75 meters (taking into account pumping during the pumping regime). From the sea side the level is 39.45 meters (theoretical null depth) with a 2.0 meter-high sea (approach of wave trough). The statistical head is 5.3 meters. The water conduit of turbine unit number 1 is drained;

b) computational case number 2 (maintenance). Level of water in the basin is 38.25 meters (pumping out of basin during pumping regime taken into account). The maximum possible statistical level of the tide of 43.5 meters and a 2.0 meter-high sea (approach of wave crest) are assumed from the sea side. Magnitude of head is 5.25 meters. The water conduit of turbine unit number 1 is drained;

c) computational case number 3 (maintenance). Level of water in the basin is 44.75 meters, level of sea is 43.50 meters. Head is 1.25 meters. The water conduit of turbine unit number 1 is drained;

d) computational case number 4 (operational), corresponds to computational case number 2 in terms of level and head.

A schematic diagram of the loads and one of the computational cases are presented in Figure 5-1.

Ice loads on the structure were not taken into account because analysis of natural conditions revealed an absence of ice floes and large blocks of ice at the approach to the TPP from the side of Ura Bay, while in the basin, the formation of ice during operation of the TPP is hampered as a result of the constant heat exchange of water with the water in the inlet and because of the fluctuating levels.

The bending moments and the normal and lateral forces were determined in five computational cross sections, chosen in the most representative sections and in areas of abrupt changes in the geometrical characteristics of the cross sections. Normal, tangential and main stresses were determined for the first (resilient) performance stage of reinforced concrete, which is warranted since tensile stresses do not anywhere exceed the rated strength of extension of the concrete of $R_p = 27 \text{ kgf per cm}^2$ (for concrete with a 400 rating).

/104/

Insignificant tensile stresses, which do not exceed 6 kgf per cm^2 , predominate over most of the length of the upper part of the block. The zero line of the stresses passes through the upper third or quarter of the block. Thus, the main part of the block is compressed by static loads.

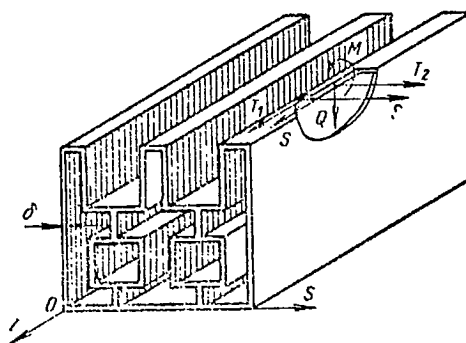


Рис. 5-2. Расчетная схема блока ПЭС как призматической оболочки.

Figure 5-2. Design diagram of the TPP block as a prismatic shell.

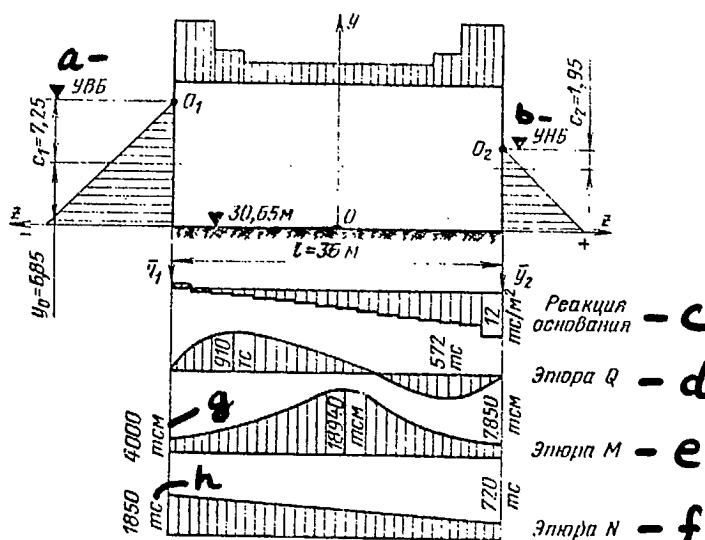


Рис. 5-3. Расчет блока как призматической оболочки. Эпюры усилий.

Figure 5-3. Design of the block as a prismatic shell. Diagram of forces.

a - head water level; b - tail water level; c - reaction of foundation;
 d - representation of Q; e - representation of M; f - representation of N;
 g - ton force meter; h - ton force.

With the aim of making the above mentioned elementary computation more precise, an attempt was made to carry it out by the method of the technical theory of shells.

In this work [2], the TPP structure was viewed as a thin-walled spatial construction of the prismatic shell type with a multiply connected cross section (Fig. 5-2). The methodology was based on V.Z. Vlasov's variational method of displacements, taking into account the deformation of displacement. The prismatic shell of the constant (along the generatrix) cross section was used as the computational scheme. Due to the closely spaced transverse bulkheads (core walls) which were spaced an average 2 meters apart, the structure was assumed to be non-deformable in the plane of the transverse cross section.

The displacements of points of the middle contour surface are found as the sum of the products of two functions, each of which depends only on one coordinate. In analysing displacement in the cross sectional plane only one member is considered because deformation of the contour of the cross section is not taken into account. The analysis of longitudinal displacement includes members which take into account warping of the cross section. /105/

By solving the system of differential equations of V.Z. Vlasov's variational method and by satisfying the limiting conditions, it is possible to find the displacements and then, using well known formulas of the theory of elasticity, the stresses can be found. Computation of the TPP block structure as a prismatic shell, with warping of the cross sections taken into account, gives a more accurate result than does elementary girder theory. Normal stresses, corresponding to warping of the cross section, comprised an average 10-20% of the stresses according to the flat cross sections hypothesis.

Figure 5-3 shows the system of loads acting on the block, and representations of the forces in cross sections. Figure 5-4 shows the normal stresses in the middle block cross section (Fig. 5-4,a-flexural stresses shown as beams, Fig. 5-4,b - warping stresses).

/106/

This problem may be generalized for the case of a shell with a piecewise constant cross section, which more closely corresponds to the peculiarities of the actual Kislaya Guba TPP structure, which has a weakened central zone.

The methodology of thermal calculations for thin-walled three-dimensional structures has not, so far, been sufficiently fully developed. Calculations for the effects of temperature are not carried out and are not covered by relevant standards in the practice of reinforced concrete and floating dock design.

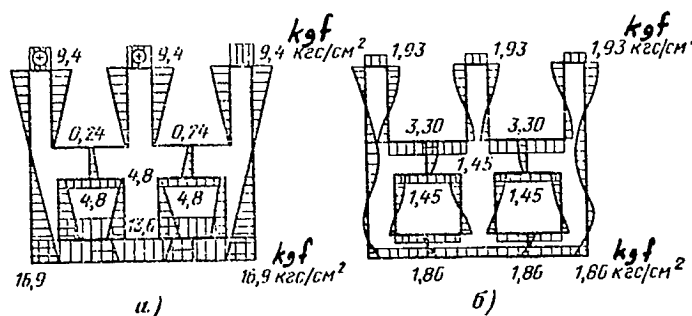


Рис. 5-4. Нормальные напряжения в среднем поперечном сечении блока.

Figure 5-4. Normal stresses in the middle cross section of the block.

Theoretical calculations and experimental research on the Kislaya Guba TPP block, however, have shown that the sharp temperature drops vertically and horizontally along the structural members give rise to significant (up to 40 kgf per cm²) tensile normal stresses.

The TPP block is divided into three zones by temperature regime: the lower zone (below the 38.7 meter mark), always located underwater and having the practically constant temperature of the surrounding medium, the middle zone within the range of fluctuations of sea and basin levels, alternately located in the open air and in the water, and the upper zone (above the 44.75 meter mark) located above the variable level zone and always in the open air. The temperature field in the upper and especially middle zones is of a non-stationary nature.

In order to establish the magnitude of the maximum tensile stresses arising in the outer members of the TPP block as a result of the above mentioned factors, the outer wall of the pier was calculated for a unidimensional (only vertical) estimated temperature drop of $\Delta t = 20^{\circ}\text{C}$. A generalization of V.Z. Vlasov's theory on thermal effects was used as the basis for the computational methodology [1]. In the work of B.S. Vasil'kov, Doctor of Technical Sciences, and E.E. Akopov, Candidate of Technical Sciences, and in the calculations carried out by the Central Scientific Research Institute of Structural Parts¹, no account was taken of the interaction of the pier plate with the transverse bulkheads or of the vertical deformation of the plate. In order to get an approximate value for the connection between the pier and the foundation plate, calculations were carried out for the two extreme computational schemes: without taking account of the contact between the wall and the bottom, and with the wall embedded in the bottom. These calculations revealed quite significant tensile stresses in the transition zone of as high as 20 kgf per cm². Research on the effect of the

/107/

¹The calculations were carried out by V.N. Kissyuk and E.E. Akopov, Candidates of Technical Sciences, and M.S. Ryabov, Engineer.

vertical compression of the wall and rigidity of the core walls of the TPP on the magnitude of the normal longitudinal stresses revealed that despite the small magnitude of vertical normal stresses (of the order of 2.5 kgf per cm^2), allowance for the deformation of the wall leads to an increase in the maximum longitudinal stresses of 11-13%. The effect of the core walls turns out to be insignificant, increasing the stresses by an average 2-3%.

The picture of the distribution of temperature stresses in the pier wall obtained by means of theoretical calculations was well corroborated by experimental data obtained by the photoelasticity method (section 5-4).

5-3. Calculations of the Static Work of the block in the transverse direction and under local loads

The static computations at right angles to the flow were in the nature of verification (verification of the sufficiency of the installed reinforcement). The flat frame in the basin part of the block was checked for the entire set of static loads acting on it.

Taking into account the geological structure of the foundation beneath the building, which consists of a layer of dense moraine 1.5-5.5 meters thick, underlain by bedrock, the calculations took into account the Winkler foundation with a variable bed coefficient. The longitudinal variables for frame member rigidity were arbitrarily replaced by constants. Calculations were carried out by Zhemochkin's method for four cases of block utilization (both water conduits filled with water, one of the two water conduits drained, both water conduits drained). The magnitude of the normal tensile stresses is 9.5 kgf per cm^2 at the bottom and $10.3 \text{ kgf per cm}^2$ in the middle pier, while the main tensile stresses are, correspondingly, 18.5 and $18.6 \text{ kgf per cm}^2$.

/108/

Calculations were carried out for the same framework for four combinations of temperatures above the deckings from the side of turbine units number 1 and 2. The following temperatures are assumed from the side of the spillway span: -20°C (no water on spillway) and $+2^{\circ}\text{C}$ (water present on spillway), the temperatures on the side of the rooms above turbine unit number 2 are $+20^{\circ}\text{C}$ and $+13^{\circ}\text{C}$.

Despite the fact that the magnitude of the thermal forces are of the same order as the static forces, and in some cases even exceed them, the stresses from the combination of the static and thermal forces only insignificantly exceed the corresponding solely static stresses (corresponding tension of 9.4 and 21.8 kgf per cm^2 at the bottom and 18.4 and 21.6 kgf per cm^2 in the middle pier).

The calculations for one of these temperature effect combinations was also carried out, using a different method, by V.N. Kissyuk, Candidate of Technical Sciences, and M.S. Ryabov, Engineer. The planar work of the framework resting on a resilient isotropic half-plane was examined. Comparison of the two calculations revealed that the use of one or the other type of resilient foundation, effects the results to a very insignificant extent, and for this reason it is possible to very significantly simplify the calculations by ignoring the joint work of the framework and the resilient foundation in the presence of temperature stresses.

Calculations of local strength were carried out for the following members: the foundation plate, external and internal pier plates, the plates of the longitudinal and transverse bulkheads, the plates of the turbine ducts, the surface spillway plate, etc. The calculations were carried out for the work of the structure as a whole, i.e. the overall strength to handle the static constructional and operational stresses.

The foundation plate was divided by longitudinal and transverse bulkheads into a series of rectilinear plates of various dimensions directly supporting the following types of operational stresses:

- from top to bottom - forces from the weight of the concrete itself and pressure on the foundation of the ballast fill;

from bottom to top - stresses from the lifting and percolating counterpressure of the water and the reaction of the foundation.

In addition, account was taken of the stresses in the plane of the plates from the general flexing of the block longitudinally. The limiting conditions for individual plates were considered in a simplified manner, with the plates assumed to be working independently of each other.

The transverse bulkheads of the ballast compartments were designed for unidirectional pressure of the water during emplacement of the block.

The other members of the TPP structure and the required reinforcement and gap width were calculated in a manner similar to the design of the plates, girders and framework.

5-4. Experimental Research

In view of the complexity of the analytical computations of the Kislaya Guba TPP block structure, a series of model studies were carried out of the stressed state resulting from static and temperature effects.

In 1964 the Laboratory of Stress Research of the Department of Water Power Utilization at the Moscow Construction Engineering Institute¹, using a three-dimensional 1:200 scale model, studied the stresses arising in the structure as the result of different temperatures in the members of the block structure (underwater part below the 40.7 meter mark +2°C, external pier walls and walls of the spillway span -20°C, production compartments +13°C) [46]. In order to study temperature stresses the method of

¹Work directed by G.L. Khesin, Doctor of Technical Sciences.

non-heated models was employed using "freezing-thawing".

Research showed that the maximum tensile stresses in the walls operate in the temperature drop zone from the spillway side, and that they attain 40 kgf per cm² in the middle transverse section. The decking over both turbine ducts is stressed, with a maximum stress of approximately 20 kgf per cm² (Fig. 5-5). By means of further refinement of the temperature drop curves (temperature changes from +2°C to -20°C in the transition section from 40.7 to 42.0 meters) it became possible to lower the stresses by 20-25%.

The effectiveness of lowering thermal stresses by means of splitting the TPP block structure by means of vertical or horizontal joints was studied experimentally with the aid of flat models. Experimental results indicate that vertical joints for the entire height of the upper zone are more effective and that when there are as few as six joints the maximum stresses are halved, while to obtain a similar result using a horizontal seam at the 40.5 meter mark, its length would have to be approximately 70% of the block length.

In 1963-1964 the Laboratory of Hydraulic and Electrical Models of the All-Union Scientific Research Institute of Transportation Construction¹ conducted research whose aim was to determine the thickness of the thermal insulation for the external surfaces of the reinforced concrete members of the Kislaya Guba TPP block which would lower to a permissible value the tensile temperature stresses in the concrete arising in the winter as the result of thermal effects on the TPP structure during changes in the water level.

¹Work directed by E.S. Luk'yanov, Doctor of Technical Sciences and M.B. Golovko, Candidate of Technical Sciences.

The research consisted in using hydraulic integrators, designed by V.S. Luk'yanov, to perform computations of the temperature regime of the external pier and the spillway plate of the TPP block, which are subjected to the effects of variable water level and, using an approximation method, to determine the tensile stresses on the surface of these members as they cooled while protected by various thicknesses of thermal insulation (and also in the absence of thermal insulation).

The calculations of irregular temperature fields were carried out for unidimensional problems, i.e. taking into account the flow of heat only along the width of the member. On the basis of experience gained in earlier reseach, the proportions of periods of time during which the external surfaces of the members are in air and under water are taken to be 11.4:1. The temperature conditions of the members were treated as for an established periodic temperature regime. /112/

The graphs plotted make it possible to determine the magnitude of the maximum tensile stresses in the members and to choose the required thickness of the thermal insulation layer. The research conducted, on the effect of transverse bulkheads on the distribution of temperatures and stresses in the structure, revealed that this effect is very insignificant and may be ignored.

The hydraulic integrator was used to determine the stress in the upper zone of the pier resulting from the drop in temperature of the surrounding air in the absence of thermal insulation. Calculations have shown that as the result of an 18°C drop in air temperature over two days, the tensile stresses arising on the external surface of the pier are of the order of 13 kgf per cm², while $R_{\text{max tensile stress}}$ is as high as 27.6 kgf per cm².

As a result of the research which was carried out, values were obtained for the necessary thickness of the thermal insulation, made out of boards treated with wood preservative, in the fluctuating level zone, which would ensure a safe magnitude of tensile stresses on the external surfaces of the reinforced concrete members, equal to $0.5 R_t = 13.5 \text{ kgf per cm}^2$ in the absence of waterproofing. This thickness is 7 centimeters for horizontal surfaces and 5 centimeters for vertical surfaces. However, in terms of weight and useful life, the use of wooden thermal insulation was deemed unsuitable and a foamed epoxy resin hydrothermal insulation, created by the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute was used at the Kislaya Guba TPP on the upper part of the structure in order to protect it from significant changes in temperature, which give rise to additional stresses in the structure. In terms of its thermal properties it is equivalent to 6-8 layer-thick wooden sheathing.

In order to precisely determine the thermal insulation effect of the sheathing it is necessary to establish the actual temperature of the surface of the concrete beneath the insulation. In order to do this four resistance thermometers (No. 690, 756, 762 and 792) were installed on the surface of the concrete beneath the thermal insulation in two locations (Fig. 5-6), which makes it possible, by comparing their readings with the temperature of the surrounding environment, to get an idea of the level of effectiveness of the thermal insulation.

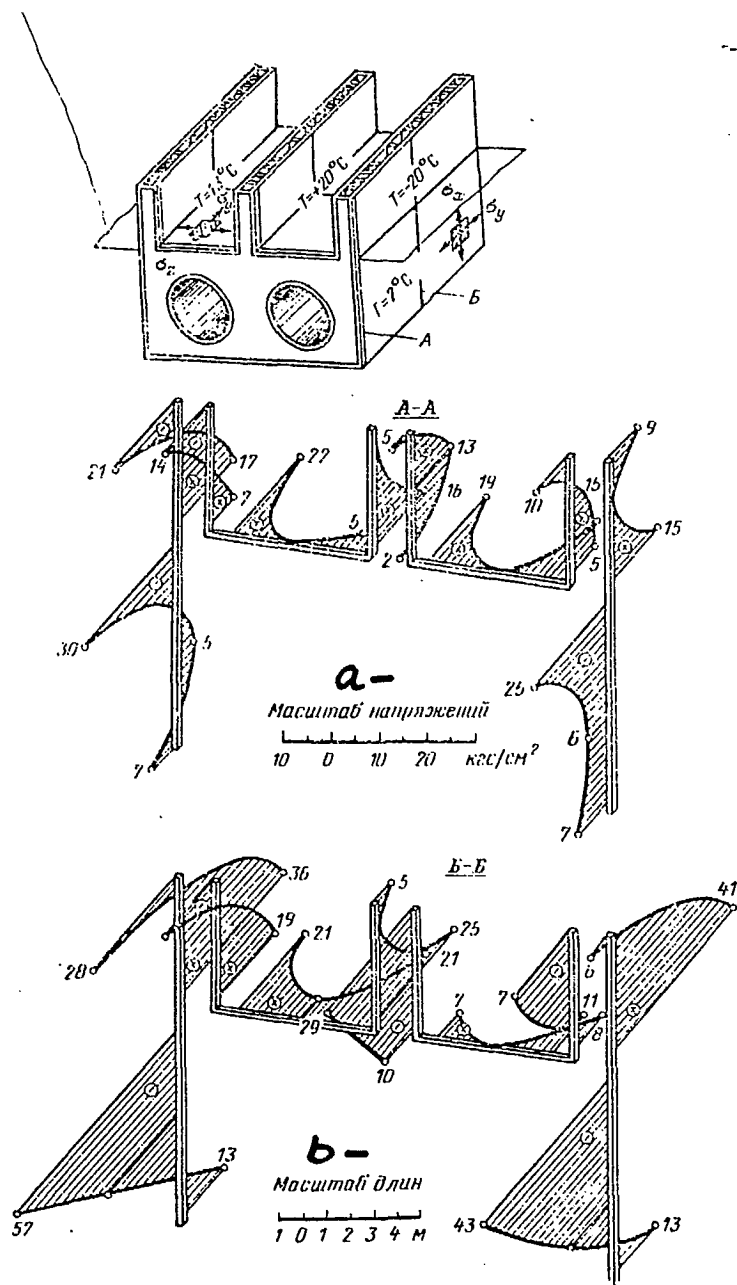


Рис. 5-5. Схема загрузки модели и эпюры напряжений в характерных сечениях.

Figure 5-5. Schematic diagram of model loads and stress curves in characteristic cross sections.

a - scale of stresses, kgf/cm^2 ; b - scale of lengths, meters.

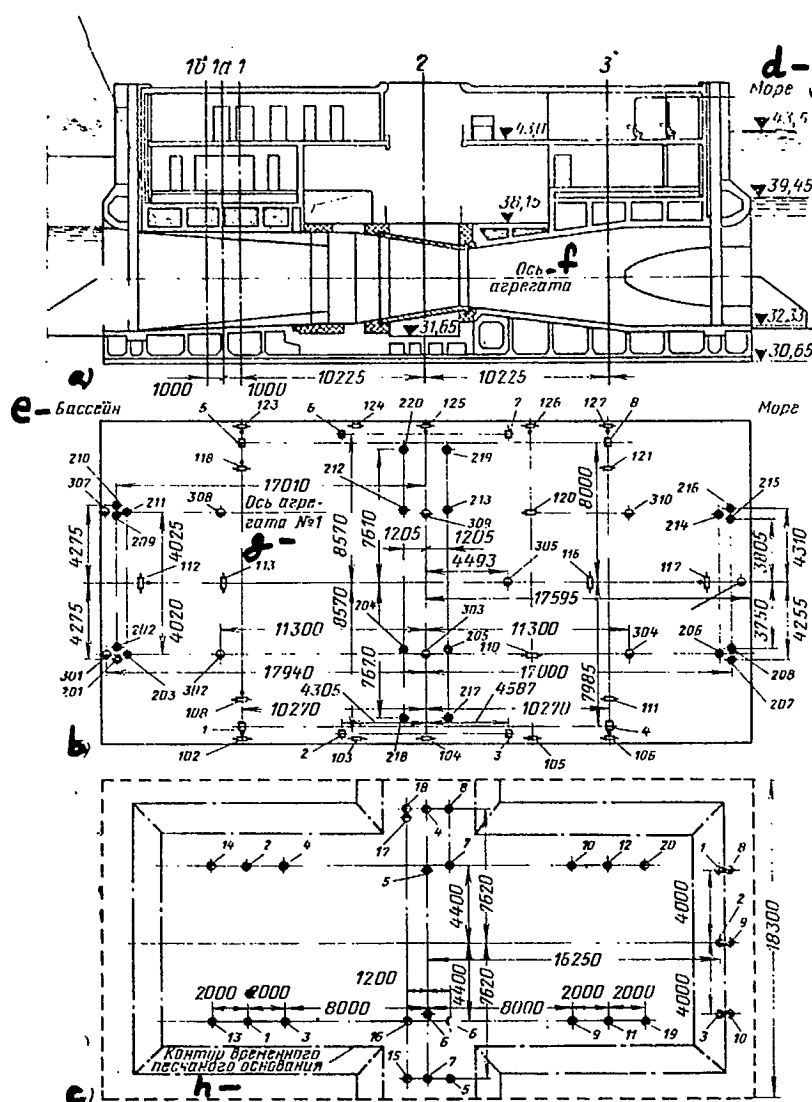


Рис. 5-6. Расположение приборов для натурных статических исследований.

a — расположение измерительных створов; б — расположение приборов в фундаментной плите; в — расположение приборов в основании блока в доке и в створе; г — арматурный динамометр (вид сбоку и с торца); 2 — грунтовые динамометры, уложенные насухо; 3 — грунтовые динамометры, уложенные под водой и в доке; 4 — грунтовые динамометры, уложенные под водой в створе; 5 — пьезометры; 6 — термометры сопротивления; 7 — пьезодинамометры.

Figure 5-6. Distribution of instruments for on-site static research.

a - distribution of measuring instrument sites; b - location of instruments in the foundation plate; c - location of ground dynamometers in the block base in the dock and at the site; 1 - armature dynamometer (side and end view); 2 - ground dynamometers, dry-installed; 3 - ground dynamometers, installed under water and in the dock; 4 - ground dynamometers, installed under water at the site; 5 - piezometers; 6 - resistance thermometers; 7 - piezo-dynamometers; d - sea; e - basin; f - turbine unit axis; g - axis of turbine unit No. 1; h - contour of temporary sand foundation.

Figure 5-6 Cont'd

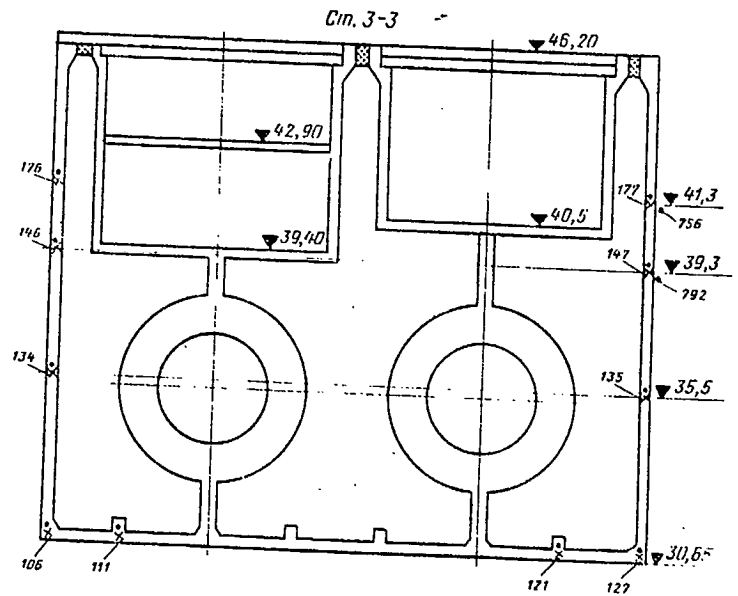
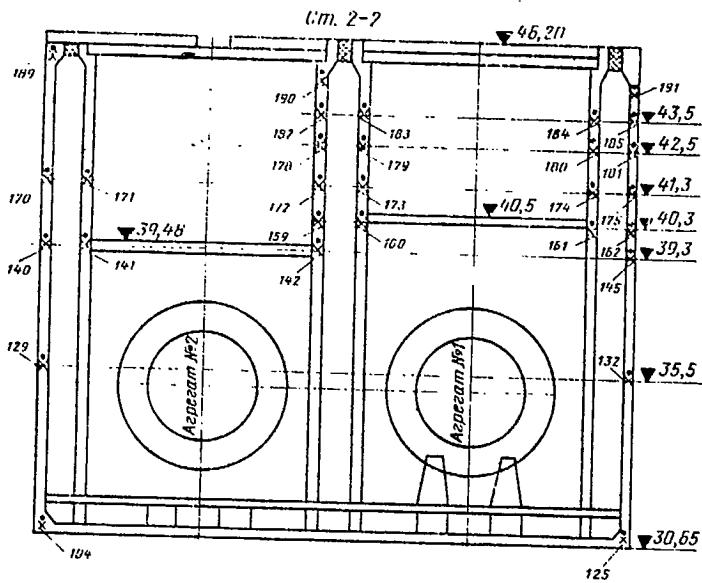
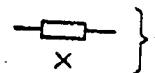
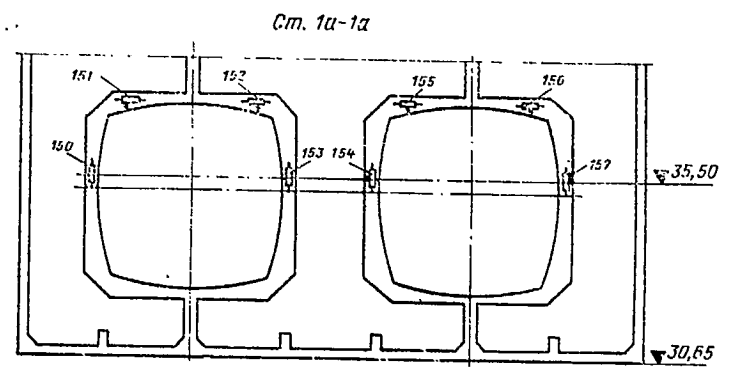
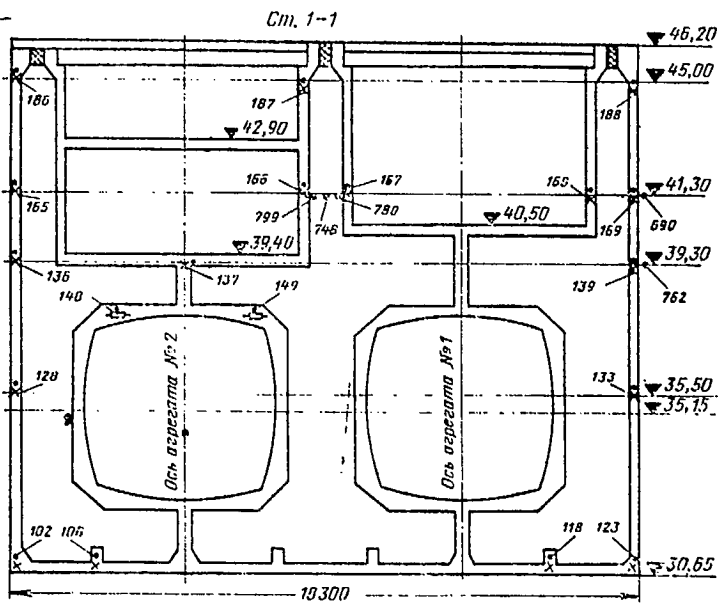


Рис. 5-6.



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Рис. 5-6.

5-5. Full-Scale Static Research on the TPP Structure

The innovative design of the Kislaya Guba TPP structure and its unusual method of construction set before the designers a whole series of problems the final solution to which could not be obtained on the basis of analysis of existing experience in erecting and operating structures similar to the TPP in either design solutions or methods of carrying out the work, or from the results of theoretical and laboratory research. Such problems include: evolution of the stressed state of the structure during its construction in the presence of the simultaneous action of various natural and technological factors and under load, the nature of the contact of the block base with the foundation during its emplacement at the site and during the subsequent operational period, the temperature regime of the structure, etc. With this aim, a series of full-scale studies were carried out at the Kislaya Guba TPP, including both the construction (1964-1968) and operational periods (1969-1970). During the construction period the research was carried out by the State Institute for Rural Power Engineering¹.

The research program, construction of the equipment used, quantity and location of instruments in the structure, and the program and performance of observations were coordinated with the designers and the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute.

Because of the fact that at the time research began (1964) the investigators did not have at their disposal the instrumentation specifically designed for use under conditions at the Kislaya Guba TPP (thin-walled reinforced concrete design, floating method of installation, presence of seawater), instruments were used which are normally employed for similar

¹The work was directed by A.I. Svetilov and R.M. Fil'roze, Engineers.

observations at river hydroelectric power plants. For this reason, one of the aims of carrying out the field observations at the Kislaya Guba TPP, in addition to those mentioned above, was also to examine the possibility of using existing instruments for similar construction.

A total of 201 instruments were installed in the structure for static research, including: 20 ground dynamometers, 79 reinforcement dynamometers, 76 resistance thermometers, 10 piezometers, 8 tubular piezometers, and 8 surface level markers. In addition, 20 ground dynamometers were installed in the interstitial block bed in the construction dock and 10 ground dynamometers in the man-made foundation at the TPP site. The distribution of the instruments in the TPP block structure is shown in figure 5-6. /117/

Investigation of the stressed and temperature conditions of the structure. The stressed state of the reinforced concrete structure of the Kislaya Guba TPP block is evaluated from results of stress measurements in the reinforcement. AD-20 reinforcement dynamometers is in the form of a steel tube within which is a wire attached at both ends, whose deformation parallels the deformation of the tube. The instrument is equipped with a device for transmitting the signal to the receiving equipment. The dynamometers were welded to the reinforcement rods after each successive concrete block was reinforced, so that the reinforcement dynamometers, being an integral part of the reinforcement, detect stresses arising within it. In order to take into account the affect of temperature on the readings of the reinforcement dynamometers, remote semiconductor resistance thermometers were installed in the concrete next to almost every one of the dynamometers. AD-20 reinforcement dynamometers, which are designed for a reinforcement rod diameter of 20 millimeters, were used in the TPP block, which has a

reinforcement rod diameter of 10 millimeters, only because dynamometers of this diameter are not available at present. There is the danger that installation of an instrument in a thin (15-20 centimeters) wall highly saturated with reinforcing rods may affect the local stressed state of the structure.

The results of theoretical calculations and experimental research were taken into account during placement of the instruments. Since the middle cross section (location 2-2, Fig. 5-6) bears the greatest load, 29 dynamometers were installed in it (27 in the pier walls and 2 in the bottom plate). From the basin side, in location 1-1, 28 instruments were installed (including 4 in the bottom plate and 11 in the walls of the turbine ducts). Ten instruments were installed on the sea side in location 3-3. Most of the dynamometers were installed longitudinally along the block. Vertically, the instruments are located in the bottom plate, at the axis of the turbine units, and at the fluctuating level zone (beginning at the 39.3 meter mark) at one meter intervals, with several instruments installed, as a rule, at each level (in the walls of the end and middle piers and in various other places). The instruments are oriented along the axis of the walls and do not sense the normal stresses resulting from warping of the plates. /118/

Monitoring of the reinforcing rod dynamometers and resistance thermometers began as soon as each block was concreted and then continued throughout the subsequent construction period. During the period of construction, monitoring of the reinforcing rod dynamometers was timed to coincide with characteristic states of the structure, i.e. mainly with the times at which there were changes in the stresses on the structure in the dock and during subsequent phases of the operations using the floating method. Monitoring of the resistance thermometers was timed, during the construction period, to coincide with characteristic changes in the temperature conditions.

Much data was accumulated during the period of field observations. Limits on the size of this book make it possible to relate the main results only very briefly. As a result of the working conditions of the block as a whole in the direction of flow, the foundation plate is one of the most stressed members of the structure. For this reason it is necessary to monitor the evolution of its stressed state during the construction period as well.

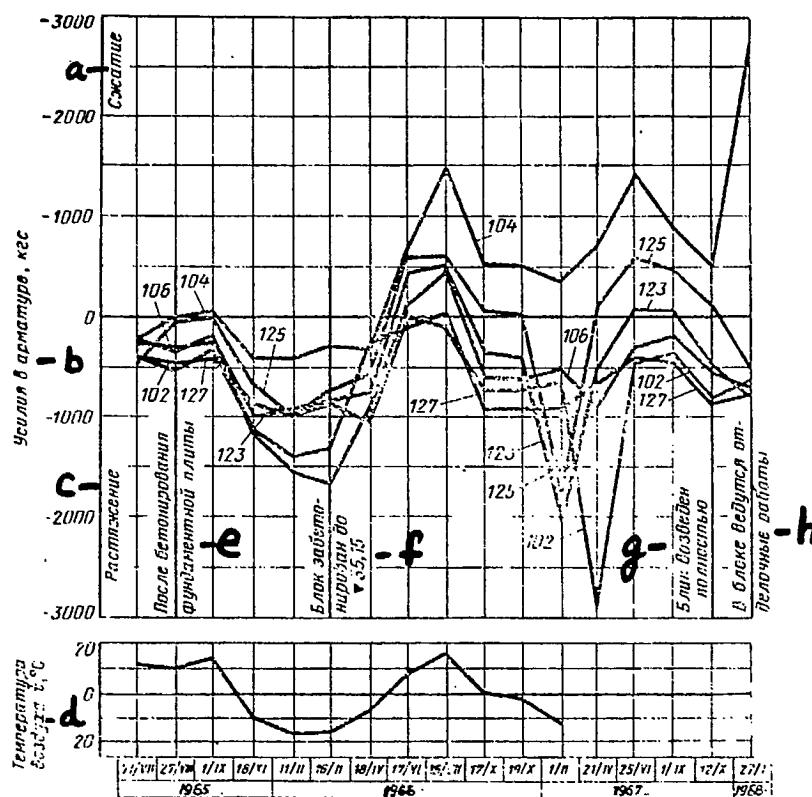


Рис. 5-7. Изменение усилий в продольной арматуре фундаментной плиты блока в ходе его возведения.

Figure 5-7. Changes in the stresses of the longitudinal reinforcement of the foundation plate of the block during its erection.

a - compression; b - stresses in the reinforcement; c - tension; d - air temperature, $t, ^\circ\text{C}$; e - after concreting of the foundation plate; f - block concreted to 35.15 meter mark; g - erection of block completed; h - finish-work being done on block.

Figure 5-7 shows curves of stress changes in the longitudinal reinforcement of the plate during erection of the block. (The figures on the curves correspond to the numbers of the reinforcing rod dynamometers shown in Figure 5-6.) The figure also shows the curve of temperature changes of the outside air. The dependence of stresses detected by the reinforcing rod dynamometers on the temperature is clearly evident in the seasonal graph. Stress changes in the reinforcement of the foundation plate due to increasing weight of the structure are insignificant. In fact, stresses recorded when the temperatures were similar on 1 September 1967 and 1 September 1965 differ insignificantly from each other.

In order to determine the effect of daily temperature fluctuations on stress changes in the reinforcement, two cycles of more frequent monitoring of the instruments were carried out. The first of these was conducted in March and April 1968. When the temperatures of the outside air were below freezing, and the second cycle in July of the same year, when the temperatures were above freezing. By this time the major work on erection of the block was completed so that no noticeable changes in the static forces occurred during the observations. Analysis of the results of the first cycle of observations reveals that changes in the temperature of the outside air noticeably affect only stresses in the longitudinal reinforcement of the block walls, which are in the zone of intermittent wetness. In these areas changes in the outside temperature from $+10$ to $+5^{\circ}\text{C}$ result in a 10-15 degree change in the temperature of the concrete and to corresponding changes in stresses of the reinforcement. As for the upper and lower zones of the walls, in these areas the same temperature fluctuations of the outside air result in a concrete temperature change of $2-4^{\circ}\text{C}$ at the 30.65 meter mark and in a change of approximately 5°C at the 41.3

/119/

/120/

meter mark. At temperatures above freezing the reaction of the reinforcement in various areas of the block walls was found to be opposite to that recorded during negative temperatures.

Measurements revealed the obvious fact that the closer an instrument is to the central axis of the block, i.e. with increasing distance from outside surfaces, the stress fluctuations detected by it as the result of external temperatures decrease.

The period of flooding of the construction area and floatation of the block is characterized by a decrease in the static forces acting on the structure. Of particular interest and importance during this process was the determination of stresses in the reinforcement of the foundation plate, inasmuch as they could indicate the formation of cracks in the concrete. Readings showed that after the foundation pit had been flooded to 8 meters, measurements at all locations showed an increase in tensile stresses of 300-400 kgf in the reinforcement of the plate. However, abrupt stress increases, indicating the formation of cracks, were not recorded and the maximum stress magnitude did not exceed 2,400 kgf. In the upper part of the block, i.e. in the walls of the end and middle piers, an increase in compressive stresses was recorded.

The settlement of the block onto its temporary foundation in the construction dock, which was required in the course of construction, made it possible, as it were, to create conditions under which the block would be at its final location. During this stage almost all of the reinforcing rod dynamometers recorded both insignificant increases (up to 120 kgf), and decreases (up to 80 kgf) in existing stresses.

While the block was being towed, despite wave action, there were practically no stress changes in the reinforcement (on the whole, stress fluctuations did not exceed 150-200 kgf). Just as during the settlement of the block onto its temporary foundation in the construction block, there was an increase in the tensile stresses throughout the reinforcement of the block walls when it was submerged at the TPP site, but their magnitude did not exceed 1,200 kgf. The maximum tensile stresses in the foundation plate were approximately 2,800 kgf as the block touched bottom.

Research during the operational period is being carried out by the All-Union Planning, Surveying and Scientific Research Institute, with the aim of:

/121/

- a) determining the stresses state of the structure during its operation as a dam;
- b) determining the effect of static and temperature factors on the formation of stresses;
- c) comparison of field data with computational results.

On the basis of the initial measurement results only preliminary qualitative conclusions can be drawn. First of all, the extremely weak dependence of stresses in the reinforcement on level fluctuations in the races should be mentioned. In view of the fact that one of the races of the structure (on the sea side) experiences constant changes in level, one would, a priori, expect similar changes in the readings from the reinforcing rod dynamometers. When the plant is not operating, i.e. when the pressure front is completely cut off, the instruments should record stress changes in the shape of a sine wave. However, this does not occur. Most of the dynamometers show reading differences during the course of one, and even two, succeeding tidal cycles of 1-2 cycles per second, i.e. 20-25 kgf

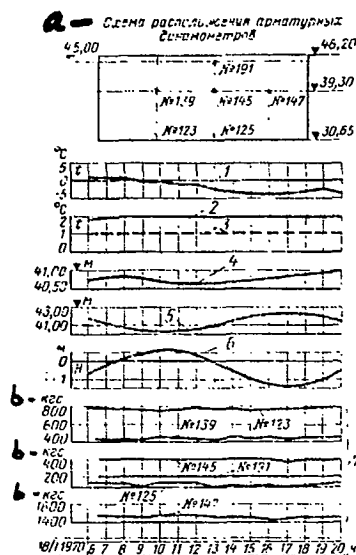


Рис. 5-8. Усилия в продольной арматуре наружной стенки бычка со стороны агрегата № 1, 18/I 1970 г.

1 — температура воздуха; 2 — температура воды моря; 3 — температура воды в бассейне; 4 — уровень воды в бассейне, м; 5 — уровень воды в море; 6 — напор; 7 — графики усилий в арматуре.

Figure 5-8. Stresses in the longitudinal reinforcement of the external pier wall from the side of turbine unit No. 1, 18 January 1970.

1 - air temperature; 2 - sea water temperature; 3 - temperature of water in the basin; 4 - level of water in the basin, meters; 5 - level of sea water; 6 - head; 7 - curves of stresses in the reinforcement; a - distribution scheme of the reinforcing rod dynamometers; b - kgf.

when converted to stresses (Fig. 5-8). In view of these insignificant changes in the readings, it would appear that the sensitivity of the AD-20 dynamometers is insufficient. In any case, their sensitivity does not make it possible to obtain very reliable correlational relationships between levels in the races and force factors. A characteristic graph of stresses in the reinforcement of the block wall from the direction of turbine unit No. 2, obtained at three measurement sites, is presented in Figure 5-9. The measurements correspond to stresses on 8 February 1970, when the maximum head on the structure was 2.9 meters (level in basin 40.15 meters, sea

level 43.05 meters). The effect of a temperature drop vertically along the wall is observed at measurement site No. 2, while no sharp changes in stresses due to a temperature gradient were recorded at sites 1 and 3, and the curves correspond to calculated curves. Figure 5-10 shows curves of the behaviour of stresses in the reinforcement of the external block wall (in the central transverse section from the direction of turbine unit No. 2) during the course of two weeks in February 1970. The same figure shows curves of changes in the mean daily temperatures of the air and the mean daily pressures on the structure (in view of the weak dependence of stresses on daily level changes, this indicator makes it possible to take into account this dependence in a generalized manner over a longer period of time). This last value is equal to 20-30 centimeters and, in view of its small magnitude, does not have to be taken into account. Thus, stress changes in the reinforcement are a consequence of temperature fluctuations of the surrounding air (all curves are qualitatively similar).

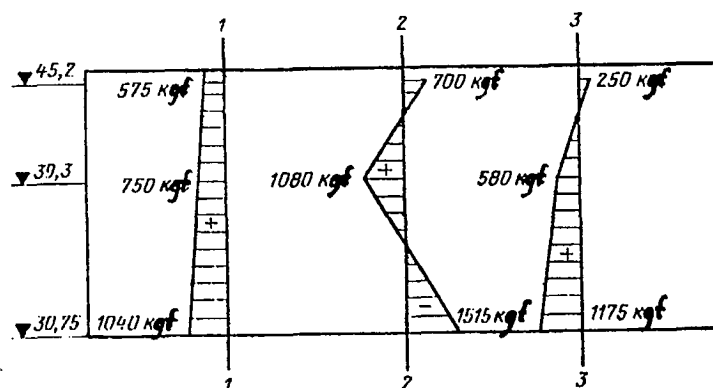


Рис. 5-9. Распределение усилий в арматуре наружной стенки крайнего бычка со стороны агрегата № 2, 8/11 1970 г.
(+) — растяжение; (-) — сжатие.

Figure 5-9. Distribution of stresses in the reinforcement of the external wall of the end pier from the direction of turbine unit No. 2, 8 February 1970.

(+) - extension; (-) - compression.

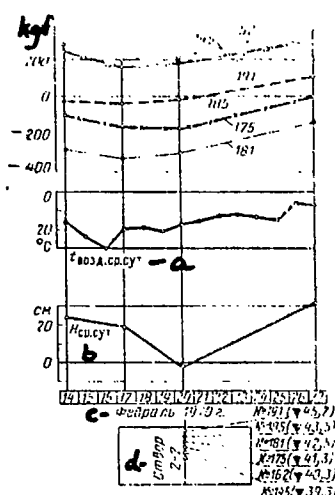


Рис. 5-10. График изменения усилий в среднем поперечном сечении наружной стенки крайнего бычка (эксплуатационный период).

Figure 5-10. Curves of stress changes in the middle transverse cross section of the external wall of the end pier (operational period).

a - mean daily air temperature; b - mean daily H; c - February 1970; d - site 2-2.

Investigation of contact stresses in the block foundation. When the floating method of construction is used, investigation of the nature of the contact between the bottom of the structure and the foundation acquires a special significance. Should the structure settle unevenly onto the previously prepared foundation one may expect both the appearance of significant stresses in the structure and the danger of the development of scouring phenomena. The closeness of the contact between the bottom of the block of the Kislaya Guba TPP and its foundation is evaluated by means of measuring the stresses at the concrete-ground contact as well as by the counterpressure of the water in the foundation. The contact stresses are measured with the aid of 20-wire membrane ground dynamometers built into the foundation plate in such a way that the surface of their membranes is located flush with the bottom of the block.

The distribution of the instruments (Fig. 5-6) was decided upon taking into consideration the following. At the time the foundation plate was being concreted the researchers had only 20 instruments at their disposal. With this in mind, they were placed in such a way that the readings from the individual instruments would give a picture of the distribution of stresses over various parts of the block bottom and also to evaluate the degree of non-uniformity of the soil structure of the artificial foundation. The instruments were located in two lines of four instruments each, transversely across the flow. The twelve instruments were placed in four groups of three instruments each, at a distance of 0.6-0.7 meters from each other, close to the basin and sea boundaries of the foundation, along the axis of turbine units No. 1 and 2.

Observations of the contact stresses were begun in May 1965. Analysis of the results of observations of the period of erection of the block in the construction area revealed that the readings of many of the instruments were incompatible and unstable. Despite the steady growth of pressure on the foundation, almost all of the instruments periodically showed sharp decreases in contact stresses alternating with equally significant increases in the stresses. On individual days many of the instruments were found to be carrying no load. The stresses recorded by the ground dynamometers did not exceed 0.8 kgf per cm^2 during this period. /124/

Instrument readings from this period do not make it possible to either elucidate some sort of quantitative relationships or even to find any clearly evident tendencies. This indeterminacy of dynamometer readings is, evidently, a consequence of a disturbance in the close contact between the instrument membranes and the sand foundation.

When the block was floated in the flooded construction area it became possible, knowing the draft of the block, to test the zero level of the instruments. The possibility of doing this, which was provided by the floating method of construction, was taken advantage of and it was found necessary to correct the calibration curves of several of the ground dynamometers.

During the time that the block rested on the artificial foundation in the construction dock, most of the instruments recorded very insignificant stresses (approximately 0.01-0.05 kgf per cm²), whereas with a block weight of 5,420 ton force, an uplift pressure, corresponding to the degree of submersion of the block, equal to 3,680 ton force, and a foundation pad area of 394 m², the stresses should have been an average of approximately 0.4 kgf per cm². The reason for this discrepancy, it seems, in addition to the displacement of the zero levels of the instruments, should again be sought in the insufficiently close contact of the instruments with the ground, due to irregularities in the micro-relief of the foundation.

This disparity between the calculated and recorded values of the contact stresses (actual values smaller than calculated values) is also being observed during the operational period. The significant scatter of the instrument readings makes it difficult to carry out a comparative analysis. Thus, for example, the average readings for 11 January 1970 (change in the readings throughout the day was insignificant) recorded by the instrument grouping No. 201, 202 and 203, were 0.83, 1.09 and 1.32 kgf per cm² respectively. As a consequence of the fact that the ground dynamometers detect the sum of the uplift pressure of the water and the reaction of the foundation itself, and since the magnitude of the latter is quite small, even a small error in the instrument readings results in significant errors when similar values are subtracted from each other. /125/

With the aim of checking the effectiveness of various methods of measuring contact stresses, ground dynamometers were also installed directly within the artificial foundation of the temporary bed of the construction dock and within the bed at the TPP site (Fig. 5-6).

Twenty instruments were installed in the soil pad in the dock, 13 dry and 7 by divers. The instruments were installed, membrane side up, at a depth of 35 centimeters from the soil's surface, and the upper soil layers were packed. Most of the instruments gave stable and matching readings. Prior to settlement of the block, the instruments showed average contact stress readings of 0.05 kgf per cm^2 . After settlement of the block, the instruments recorded a rise in stresses to 0.1-0.2 kgf per cm^2 . This was less than the expected values of 0.4 kgf per cm^2 but much closer to it than the readings of the dynamometers built into the bottom of the block. A small decrease in stresses was observed after the block was floated off the pad and, after the block was submerged again, the stresses increased to 0.2-0.3 kgf per cm^2 .

Ten ground dynamometers (Fig. 5-6) were installed in the gravel-sand foundation of the TPP block, which was installed underwater by divers, however, after the bilge blocks were dragged out, the cables of seven of the instruments were found to be damaged and the remaining three instruments could not be found by the divers.

The experience of carrying out field observations of the reactive stresses of the ground makes it possible to conclude that installation of ground dynamometers in the bottom plate of the structure did not provide sufficiently satisfactory results. The most complex problem in this regard is to ensure constant contact of the instrument membrane with the surface of the ground (with the exception of those cases, of course, when the detachment of the bottom of the structure from the ground is stipulated by its operating conditions).

Installation of instruments directly into the body of the foundation is, in this sense, more reliable, but it requires special difficult measures to be carried out during the course of underwater operations.

5-6. Field Studies of the Stressed State of the TPP Structure as the Result of Dynamic Interactions

Quite comprehensive theoretical and experimental studies of the static work of the Kislaya Guba TPP structure were carried out during its design stage (sections 5-2 to 5-4), but the nature of the structure's operation in the presence of dynamic interactions was not studied. In view of the fact that elucidation of the operating conditions of the new thin-walled design in the presence of periodic and occasional dynamic loads would be of significant interest, a series of dynamic full-scale studies were carried out at the Kislaya Guba TPP by hydraulic and dynamic research sections of the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute¹ [33]. Their aim was to:

/126/

1. study the hydrodynamic loads on the members of the water conduit under various operating regimes of the TPP.
2. determine the dynamic properties of the structure and its reaction during interaction with the flow.

In accordance with the research schedule, the forms and frequencies of the natural oscillations of the block in the construction area at Cape Prityka were studied in March and April of 1967. By that time the block had been erected up to the design height and the spacing chambers were empty of ballast. The dynamic characteristics of the structural members of the surface spillway plate, the end pier from the side of turbine unit No. 1, and the turbine chamber of unit No. 2 were studied throughout this time.

¹Work carried out under the supervision of V.M. Lyatkher, Doctor of Technical Sciences, Executive Director - Yu.I. Braslavskii, Engineer.

The natural oscillations of the spillway plate were generated by means of a 32.5 kgf weight, falling onto it from various heights of 10-550 centimeters. The oscillations of the spillway plate and ribs were recorded by vibrographs located on the surface of the plate and in the spacings on the vertical ribs and decking of water conduit No. 1 (Fig. 5-11).

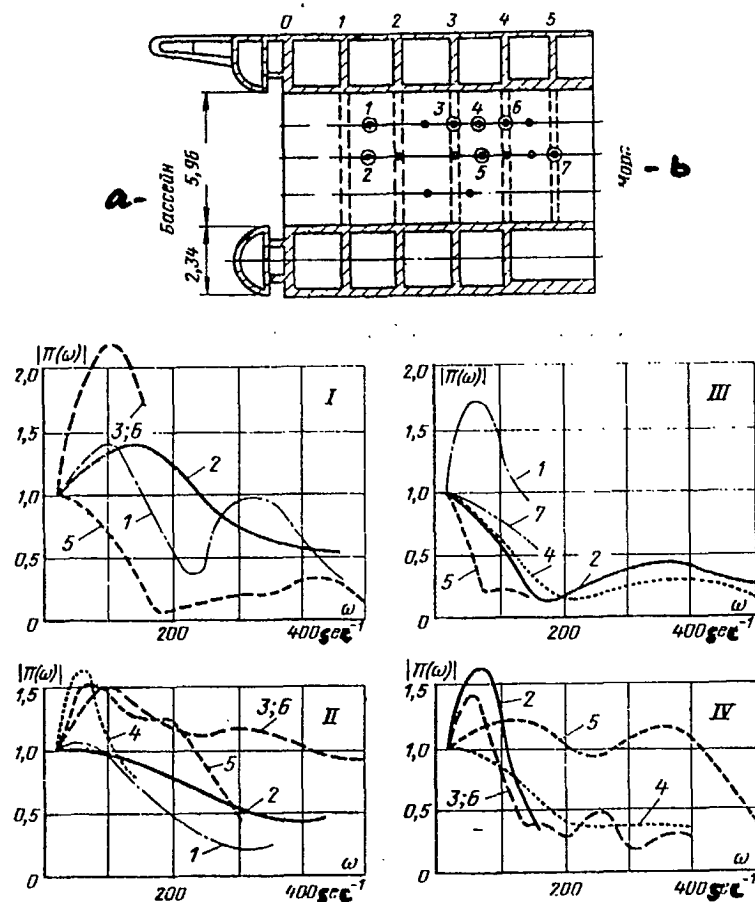


Рис. 5-11. Размещение датчиков и точек возбуждения колебаний на плите водослива. Графики передаточных функций.

Номера на кривых соответствуют точкам возбуждения колебаний.
 ● — точки возбуждения колебаний; ○ — точки регистрации колебаний; I — модули передаточных функций перемещений центра плиты; II — то же середины пролета продольного ребра; III — то же середины пролета поперечного ребра; IV — то же пересечения ребер.

Figure 5-11. Distribution of sensors and vibration generation points on the spillway plate. Transmission function curves.

The numbers on the curves correspond to vibration generation points.

a - vibration generation points; ● - points at which vibrations were recorded; I - transmission function moduli of the displacement of the center of the plate; II - same for centre of longitudinal rib span; III - same for center of transverse rib span; IV - same for intersection of ribs; a - basin; b - sea.

Free oscillations of the pier were generated by a concrete block weighing 1,750 kgf, suspended from the hook of a tower crane. The freely hanging weight was at a distance of 5-10 centimeters from the wall. In order to generate the vibrations, the weight was pulled 1-2 meters away from the wall. Vibrations of the pier were generated by blows at 4 points. During this time the vibrographs were placed in such a way as to obtain information about vibrations in various directions of both the pier as a whole, and of the structural members associated with it. /127/

Vibrations of the reinforced concrete construction of the turbine water conduit of unit No. 2 were generated by dropping a 32.5 kgf weight from a height of 0.8-4.0 meters, and recorded by seven vibrographs, two of which recorded the horizontal and vertical components of the vibrations of displacement of the lower conduit plate while the remaining three were installed in the spacings. /128/

In order to record the vibrations a standard K001 assembly was used, consisting of three low-frequency induction (natural frequency 1 cycle per second) I001 vibration detectors, a three-channel shunting attachment of the R001 type for varying the power of the signal going to the recorder, and a set of low-frequency (natural frequency 25 cycles per second) integrating galvanometers for the loop oscillographs. The K001 assembly makes it possible to measure vibrations within the frequency range 1-200 cycles per second with amplitudes of 0.05-3 millimeters.

Oscillograms of oscillations of the structural elements were obtained during the experiment and used for computing transmission functions of displacement.

The use of transmission function (influence function), which determines the reaction of a dynamic system at some point to the action of a periodic load with a single amplitude applied at some other point of the system is convenient during analysis of the effect of stationary occasional loads on the structure.

For oscillations generated by a single-point space-time force pulse, the transmission function of displacement is an integral Fourier transform means, applied to the experimentally determined oscillation curve. When the transmission function of displacements is known, it is possible, by appropriately differentiating it, to obtain the transmission functions of the bending moments and the transverse forces. An example of the transmission functions of displacement of separate points on the spillway plate is presented in Figure 5-11.

During analysis of the calculated transmission functions, greatest attention was devoted to the frequencies of the natural oscillations of various members of the structure. Analysis of the experimental data makes it possible to quite reliably isolate the three lowest oscillation frequencies. Thus, for example, for the center of the spillway plate, the frequencies of the first, second and third forms of oscillations are 5.55-6.35, 9.6 and 16.9 cycles per second respectively, while for the spillway plate beneath the center of the longitudinal rib they are 10.3-12.7, 15.9-19.1 and 33.3 cycles per second. The spillway plate as a whole oscillates at frequencies corresponding to the first, second and third forms of oscillations, equal to 9.6-12.7, 26.9-33.3 and 57.2-60.3 cycles per second.

/129/

The cap of the end pier performs free horizontal oscillations with frequencies of 3.2, 9.85-10.8 and 19.7-23.5 cycles per second.

In the vertical plane the pier cap vibrates with frequencies of 3.82, 7-7.16 and 10.2-14 cycles per second.

The lowest natural frequencies were recorded for the abutment cap: 1.59 and 7.64-8.6 cycles per second.

The dynamic modulus of elasticity of the reinforced concrete structure was determined during testing. Its average value was 10^5 kgf per cm^2 .

These field studies of the Kislaya Guba TPP block make it possible to conclude that the block is a rigid structure whose members have lower form oscillation frequencies within the range of 3-90 cycles per second. By virtue of the rigidity of the structure under the influence of low-frequency (not exceeding 80 cycles per second) concentrated loads, load amplitudes of one ton-force result in displacements which do not exceed 4×10^{-3} centimeters.

Investigation of hydrodynamic loads on the spillway plate and members of the turbine water conduits were carried out from December 1968 through October 1969 [33]. The loads were measured with the aid of pressure pulse strain gauges built by the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute in a setup with a TA-5 amplifier and an N-700 oscillograph. The placement of the instruments is shown in Figure 5-12. The diameter of the operating portion of the instrument membrane is 30 millimeters, the frequency of the natural vibrations of the membrane is 3000 cycles per second. The strain gauge can register pressure pulses with an amplitude of 1-2 centimeters.

Testing was carried out with water flows both from the basin to the sea and in the opposite direction at heads of 0.5 to 2.7 meters. Fluctuations of the hydrodynamic pressure were recorded for various combinations of discharges through the openings in the dam (turbine unit, bottom weir, surface spillway).

Results of observations make it possible to conclude that pressure fluctuation, once discharge has stabilized, is a steady-state occasional process with a distribution close to normal. Correlation functions and pressure fluctuation spectra were calculated from the recorded pressure fluctuation data.

The maximum recorded value for pressure dispersion is 0.32 meters of the water column.

The results of field observations make it possible to conclude that the fluctuation of hydrodynamic pressure is a low-frequency process with a maximum energy within the frequency range of 0-10 seconds⁻¹. Pressure fluctuation spectra for the spillway pier also have a maximum in the frequency interval 50-70 seconds⁻¹. Computation of the reciprocal correlation function for the two sensors shows that the dimensions of the eddies which generate the pulses are of the same order as the transverse dimensions of the water passage openings.

The value of the dynamic response factor was calculated for the center of the spillway plate. As is well known, this factor is equal to the relationship between stresses in a structure under conditions of dynamic load and similar stresses under a static load condition. Its magnitude was found to be 1.004, which indicates the possibility of disregarding dynamic phenomena in the design of this member of the structure.

In order to evaluate the above design a general comparison was carried out of the dynamic properties of two hydroelectric power plants equipped with horizontal bulb-type hydraulic turbine units: the Kislaya Guba and Kiev plants. A comparison of the structures, in terms of natural oscillation frequencies, reveals that the frequencies ω_1 of the first form for the Kislaya Guba TPP are 50-65 seconds⁻¹, while for the Kiev hydroelectric power plant they are 20-25 seconds⁻¹. Similarly dimensionless

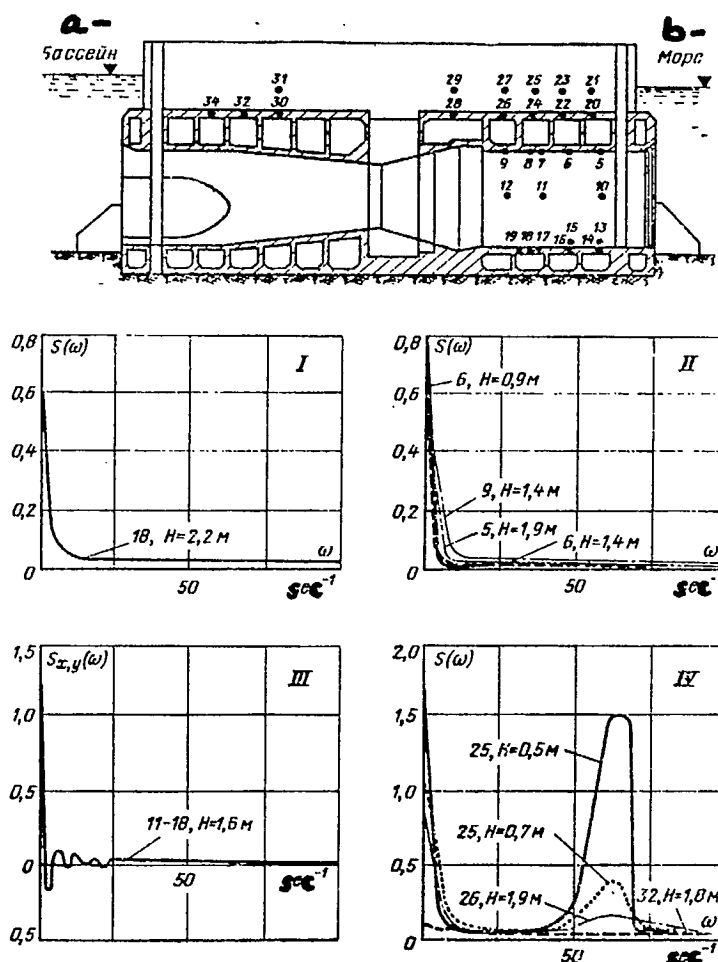


Рис. 5-12. Расстановка датчиков пульсации давления и гидродинамического воздействия на элементы водопропускного тракта ПЭС. Направление потока бассейн — море. Цифры при точках и на кривых означают номера датчиков пульсации давления. I — нормированный взаимный спектр пульсации давления, пропуск расхода через донный и поверхностный водосбросы; II — нормированные спектры пульсации давления, пропуск расхода через донный водосброс; III — нормированный спектр пульсации давления, пропуск расхода через донный водосброс; IV — нормированные спектры пульсации давления, пропуск расхода через поверхностный водосброс.

Figure 5-12. Distribution of pressure fluctuation sensors and hydrodynamic action on the elements of the TPP water conduit. Direction of flow, basin-sea. Numbers near dots and on curves indicate numbers of pressure pulse sensors.

I - standardized reciprocal spectrum of pressure fluctuation, discharge through bottom weir and surface spillway; II - standardized spectra of pressure fluctuations, discharge through bottom weir; III - standardized spectrum of pressure fluctuations, discharge through bottom weir; IV - standardized spectra of pressure fluctuations, discharge through surface spillway; a - basin; b - sea.

natural frequencies of the first form of oscillations ν_1 are found at the Kislaya Guba TPP within the range 33.5-43.6 and at the Kiev hydroelectric power plant within the range 22.0-27.2. Thus, field studies show that the Kislaya Guba TPP block is a relatively more rigid structure. In the range of perturbing frequencies generated by the passage of water through the TPP site, vibrations of the structural members are negligibly small.

The floating structure of the Kislaya Guba Experimental TPP is a unique design for hydroelectric power plants, whose strength derives from the three-dimensional work of its thin-walled members. Due to the absence of general methods for calculating such designs, a series of theoretical and model studies were carried out, which made it possible to outline the procedures for calculating the stressed state of the block under various operating conditions of the structure and to determine the quantity and distribution of the design reinforcement.

/132/

The main result of the above mentioned research is that it made it possible to provide a theoretical basis for the new progressive, reinforced concrete design, which is built up of significantly thinner members than those used in the construction of hydroelectric power plants.

The results of field observations of the stressed-strained state of the block confirm the complete reliability of operation of the TPP structure in the presence of the calculated static and temperature loads.

In the course of calculations of the Kislaya Guba TPP there was confirmation of the conjecture that the absence of expansion-contraction seams results in the appearance in the structure, which has rather large

dimensions (36.0 x 18.3 x 15.35 meters), of significant thermal stresses. A certain fund of experience has been accumulated in the application of modern methods of structural design involving elasticity theory, which will, in future, facilitate the process of designing industrial TPPs.

The results obtained cannot be considered exhaustive in view of the complexity of the three-dimensional work of the structure. For this reason, in order to test existing methodology, for its continued development, and to obtain a more precise knowledge of the actual stresses, continuing study was undertaken in 1965 of the stressed-strained state in the field (first during the construction period and then during operation).

The control and measuring apparatus which have been installed make it possible to investigate in detail the stressed state of the most important members and most highly stressed zones of the structure, which are due to static and dynamic loads.

Chapter 6

BUILDING MATERIALS

6-1. Concrete of Increased Frost Resistance

The choice of concrete as the main building material for the TPP structure was one of the most complex problems in view of the prevailing opinion about the unavoidable deterioration of reinforced concrete under the conditions of the northern tidal seas. This opinion was based on the existing example of the deterioration of the moorage at Murmansk (the reinforcing rods in all of the piles were exposed). The reason for the deterioration, as shown by analysis, were the numerous fluctuations in level caused by tides at temperatures below freezing, the corrosive action of sea water, and biological fouling. In the construction region of the Kislaya Guba TPP there are an average 200 days per year with below freezing temperatures (down to -35°C). Four hundred freezing and thawing cycles may occur during this period as the result of tidal fluctuations with an amplitude of 1-4 meters. Sea water contains more than 2,500 milligrams per liter of sulfates, has a salinity of 34 parts per hundred, and biomass encrustation may attain 5-6 kilograms per m^2 . /133/

Despite the prevailing opinion, however, on the basis of research [47a] which showed the possibility of obtaining highly frost resistant concrete (up to 3,600 cycles), work was begun on creating a durable concrete for tidal power plants and, in particular, for the Kislaya Guba TPP.

Various institutes were involved in the creation of a highly frost resistant concrete (the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, the Central Scientific Research Institute of Structural Parts, the All-Union Scientific Research Institute of Hydraulic Engineering, the All-Union Scientific Research

Institute of Hard Alloys, the Leningrad Marine Scientific Research and Design Institute, the Moscow Highway Institute, and the Scientific Research Institute of Concrete and Reinforced Concrete). The participation of these institutes made it possible to utilize all of the experience accumulated up to that time on frost resistant dense concrete for shipbuilding, marine and other special types of concrete [12, 30, 36], and to invest this research with a definite goal. For this reason, the work of creating the concrete for the Kislaya Guba TPP was also reviewed at centers coordinating work on the prevention of concrete corrosion (a section of the Scientific and Technical Council of the State Committee of the Council of Ministers, USSR for construction in 1964-1966 and the All-Union Conference on the Prevention of Concrete Corrosion in 1963).

The most important criterion in the selection of concrete for the Kislaya Guba TPP turned out to be its frost resistance rating.

Calculation of the required frost resistance rating of the concrete was carried out using a method proposed by the Central Scientific Research Institute of Structural Parts, according to which evaluation is first made of the severity of the climatic effects during standard cycles of freezing and thawing and then corrections are introduced in the form of conventional coefficients which reflect the influence of the age of the concrete, solar radiation, and interruptions in freezing. The required frost resistance rating of the concrete is then determined, with account taken of the coefficient of its homogeneity in terms of frost resistance. Finally, its frost resistance margin coefficient is assigned by analogy with the margin coefficient employed in the design of the load-bearing aspects of the structure. /134/

In assigning the frost resistance rating the important additional requirement is introduced that, by the end of testing for the stipulated number of cycles of freezing and thawing, the concrete samples show no decrease in initial strength.

In a general form this method may be expressed as:

$$F = C \frac{1}{K_H} K_I K_P K_R K_M$$

where F - is the normed frost resistance of the concrete, expressed in standard cycles of freezing and thawing with no loss of strength in the samples tested; C - is the number of standard cycles of freezing and thawing, equivalent to the number of annual natural cycles under the given conditions; K_H - is the coefficient of heterogeneity of the concrete in terms of its frost resistance; K_I - is the coefficient which takes into account the increase in frost resistance of the concrete in summer; K_P - is the coefficient which takes into account the changes in frost resistance resulting from the period of preliminary hardening; K_R - is the coefficient which takes into account solar radiation; K_M - is the margin coefficient.

The number of cycles which the concrete is actually subjected to, with the micro-climate taken into account, is 240. The datum of natural cycles, characterized by different freezing temperatures, is achieved by using conversion coefficients determined by research at the Central Scientific Research Institute of Structural Parts. It was shown that the following conversion coefficients could be employed for converting cycles of various temperatures to freezing at a temperature of -15°C . Thus, a cycle at a temperature of -2 to -5°C is equivalent to 0.17 of the above cycle, a cycle at a temperature of -6 to -10°C is equal to 0.64 while at lower temperatures it is equal to 1.0.

In recalculating the total number of natural annual cycles to an equivalent number of cycles normed to the most severe cycles (freezing temperature 11°C and below), 108 such arbitrary normed cycles was obtained for a year of average severity.

Special parallel tests, conducted at the Central Scientific Research Institute of Structural Parts, on achieving frost resistance by means of an accelerated method, and in accordance with All-Union State Standard 4800-59, showed that a single normed cycle is equivalent to the effect of three standard cycles. /135/

In rough calculations one may disregard the coefficients K_I , K_P and K_R since they do not exert a deciding influence on the degree of frost resistance. The magnitude of K_H may be taken to be 0.35 while the total margin coefficient, by analogy with calculations of the strength of hydraulic power plant structures, is taken to be equal to 2.0.

Thus,

$$F = 300 \frac{1}{0.35} \approx 1,700.$$

Taking into account the requirement mentioned above (no loss of strength), the required frost resistance rating during standard testing must be no less than Mrz 2000, which is much higher than the top All-Union State Standard 4795-68 frost resistance rating of Mrz 500. The work carried out at the Central Scientific Research Institute of Structural Parts, All-Union Scientific Research Institute of Hard Alloys, and the Scientific Research Institute of Concrete and Reinforced Concrete has shown that the achievement of such a level of frost resistance is completely feasible.

Selection of the concrete mix. A number of institutes were given the task of developing a concrete which would satisfy the above requirements. Tests of the frost resistance of proposed concrete mixes were carried out in the freezing basin of the Murmansk Weather Station of the All-Union Scientific Research Institute of Transportation Construction, using an accelerated method which made it possible, during a single winter, to obtain more than 700 cycles with freezing in air (by increasing movement of air in the basin with the aid of fans) and thawing in the sea water of Kola Bay.

The test samples were prepared using sulfate-resistant portland cement with various admixtures and aggregates, designated for the concrete to be used in the Kislaya Guba TPP (Table 6-1). In early 1964 these samples were placed into the freezing basin for frost resistance testing using the accelerated method. A reinforced concrete fragment of a TPP block, fashioned in the form of an empty box 1.3 x 1.3 x 1.15 meters, made out of concrete mix number 4, was also placed into the basin for testing at the same time. The empty space of the fragment was filled with hydrophobic soil and three side surfaces were covered with hydrothermal insulation.

The tests revealed that samples without admixtures (mix number 1) deteriorated after 300 cycles of freezing and thawing, deterioration occurred after 400-500 cycles¹ when sulfite-alcohol distillery waste (SADW) and gaseous silicone liquid (GSL) were added (mixes number 2 and 5), and after 1,540 cycles when FeCl_3 plus SADW were added (mix number 6). With the latter mix, one of the samples deteriorated and two of the samples showed a significant decrease in the dynamic modulus of elasticity after 2,000 cycles. Samples composed of mixes number 3 and 4, which contained an

/136/

¹In mix number 2 the addition of GSL was considered to be non-optimal.

admixture of air-entraining neutralized tar (AENT), did not deteriorate after 1,540 cycles (evaluated in accordance with the method of S.D.

Shestopero - 10 points on the scale) and their strength increased. There was also no evidence of deterioration on the surface of the concrete fragment composed of this concrete mix.

Table 6-1.

Concrete mixes field-tested for frost resistance (by the All-Union Scientific Research Institute of Transportation Construction Methods).

Mix Number	Composition of concrete mix per m^3 , kg				Water-cement ratio	Admixture	Admixture content, % weight of cement	Placeability of the concrete mix, seconds	Mix developed by
	Cement	Sand	Crushed Stone	Water					
1	420	700	1 220	164	0.39	None	0	30	Standard
2	420	700	1 245	155	0.37	SADW	0.1	20	All-Union Scientific Research Institute of Transportation Construction
3	420	600	1 170	151	0.36	AENT	0.06	30	All-Union Scientific Research Institute of Transportation Construction
4	420	600	1 250	151	0.36	SADW + AENT	0.1 + 0.03	25-30	All-Union Scientific Research Institute of Transportation Construction
5	410	700	1 220	164	0.39	GSL	0.06	20	Scientific Research Institute of Concrete and Reinforced Concrete
6	420	700	1 220	-	-	$\text{FeCl}_3 + \text{SADW}$	-	-	All-Union Scientific Research Institute of Hard Alloys

Note. SADW - sulfite-alcohol distillery waste; AENT - air-entraining neutralized tar; GSL - gaseous silicone liquid

The test results made it possible to make a preliminary choice for the TPP job of the concrete mix developed by the All-Union Scientific Research Institute of Transportation Construction (number 4) and to begin testing it further under both laboratory and industrial conditions in order to determine the precise composition of the mix using local materials, and to work out the special techniques for the thin-walled TPP structure¹.

The high frost resistance of the chosen mix was a result, primarily, of the organization of the concrete structure, with closed microscopic air pockets entrained in the concrete mix with the aid of the admixture, and as the result of appropriate concreting techniques. /137/

According to research carried out by the All-Union Scientific Research Institute of Transportation Construction, the high frost resistance was achieved by the entrainment of no less than 2% air in the placed concrete mix by means of air-entraining neutralized tar in amounts of 0.02-0.03% of cement weight and the plasticizing addition of sulfite-alcohol distillery waste in amounts of 0.1-0.15% of cement weight in order to increase the workability of the mix, by decreasing the amount of water, and by increasing the setting times of the mix. Comparison of the frost resistance of concrete with air randomly entrained (2.3%) during preparation and placement and concrete with admixtures of SADW and AENT reveals that concrete with air-entraining admixtures maintains its weight after 1,300 cycles of freezing and thawing and shows no decrease in its dynamic modulus of elasticity, while concrete with randomly entrained air begins to deteriorate from its exterior after 1,300 cycles and loses 4% of its weight. The expenditure of 420-500 kilograms of cement in concrete with

¹Selection of the concrete mix for the TPP and its testing were carried out by the Concrete Corrosion Laboratory under the direction of F.M. Ivanov, Doctor of Technical Sciences.

admixtures of AENT and SADW, in comparison with weight changes of the samples and their dynamic modulus of elasticity values, does not affect frost resistance.

On the basis of long-term research over many years at the All-Union Scientific Research Institute of Transportation Construction, it was shown that, to ensure high frost resistance, the quantity of water must be limited to between 160 and 180 liters per 1 m^3 of concrete mix with a water-cement ratio not greater than 0.40. Research at the All-Union Scientific Research Institute of Hydraulic Engineering also showed the need to limit the quantity of water in a concrete mix in order to ensure high frost resistance of concrete. A low water-cement ratio results in a poorly workable concrete mix (slump of a standard cone 0.4 centimeters, stiffness 25-40 seconds).

As a result of the research, in October 1964 the concrete mix proposed by the All-Union Scientific Research Institute of Transportation Construction was accepted for the horizontal members of the TPP structure. A shipbuilding concrete was recommended for the vertical members of the structure but, in accordance with the requirements of the Scientific Research Institute of Concrete and Reinforced Concrete, its use was restricted to constantly submerged areas. For the concrete in the zone of periodic inundation, the All-Union Scientific Research Institute of Transportation Construction had the task of altering its proposed mix while maintaining a low water-cement ratio in order to ensure its high workability during concreting of the vertical members and the construction of complex configurations. Such a mix was developed and, after testing, was accepted with minor changes (having to do with industrial and weather conditions) for concreting all blocks of the TPP structure in the underwater and variable level zones.

Composition of 1 m³ of concrete mix by dry weight of materials

Sulfate-resistant portland cement, 300 grade (State Standard 10178-62)	480 kg
Water	Not more than 180 liters (w/c ratio = 0.375)
Sand without organic admixtures or mica, not more than 1% impurities	630 kg
Granite-gneiss crushed stone with a solid-body-volume weight of 2.9-3.1 ton/m ³ , not more than 1% impurities:	
5 to 10 mm fractions	425 kg (35%)
10 to 20 mm fractions	785 kg (65%)
SADW	0.1% of cement weight
AENT	0.2-0.3% of cement weight
Air	50-80 liters

The cement, crushed stone and sand were measured out by weight, the water and admixtures by volume. The workability of the concrete during its placement to form the block was 1-3 centimeters slump of the cone, and the volume weight of the consolidated concrete mix of 2.54-2.60 grams per cm² was not below the rated volume weight.

The composition of the concrete mix and its main characteristics were strictly controlled during concreting (section 9-2).

Testing of the concrete in the structure. In the summer of 1968, after flooding the dock construction pit, careful observation of the condition of the concrete was begun. Tests were carried out on the concrete surface of the 15-centimeter wall, which was not protected by an insulating layer in the fluctuating water level zone on the sea side, and of the concrete surface near the bottom, which was under the greatest pressure.

The moderating effect of the warm body of water near the structure and the heat inertia of the structure itself was taken into account in determining the actual number of natural alternating cycles of freezing and thawing which affect the concrete. It was found that the moderating effect of the sea extended to a distance of 80-100 meters from the structure and that its influence was particularly strong in the zone of variable water level during ebb, where the coefficient of the micro-climate, which is equal to the relationship of the air temperature at the water surface near the structure to the air temperature at a distance of 100 meters from the structure, changes from 1.0 (when water freezes) to 0.7, when the temperature of the water changes from -1.75 to 6°C . It was established at this time that, by virtue of the micro-climate, the temperature of the concrete surface does not fall below -18°C even when the air temperature is -30°C .

/139/

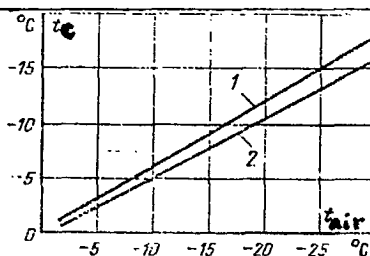


Рис. 6-1. Зависимость минимальной температуры поверхности бетона t_c в зоне переменного уровня при отливе от температуры воздуха $t_{\text{возд}}$ на расстоянии более 100 м от сооружения.

1 - при температуре моря более 1.5°C ; 2 - при температуре моря менее 1.0°C .

Figure 6-1. Minimum temperature of the surface of the concrete t_c in the variable water level zone during ebb as a function of the air temperature t_{air} at a distance of more than 100 meters from the structure.

1 - when the temperature of the sea is above 1.5°C ; 2 - when the temperature of the sea is below 1.0°C .

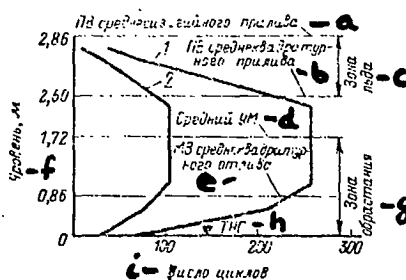


Рис. 6-2. Число циклов замораживания и оттаивания, действующих на бетон в зоне переменного уровня.

1 - число циклов, фактически воздействующих на бетон; 2 - то же расчетных (по методике ЦНИИС).

Figure 6-2. Number of freeze and thaw cycles which affect the concrete in the variable water level zone.

1 - actual number of cycles affecting the concrete; 2 - estimated number of cycles (All-Union Scientific Research Institute of Transportation Construction Methods); a - mean syzygial flood high water; b - mean quadrature flood high water; c - ice zone; d - mean sea level; e - mean quadrature ebb low water; f - level, meters; g - fouling zone; h - theoretical null depth; i - number of cycles.

Taking into account the relationship, established by observations, of the concrete temperature on the temperature of the air and water (Fig. 6-1), as well as the freezing and thawing rates of the concrete under the conditions of the Barents Sea and the temperature of the air and water in the region of the Kislaya Guba throughout the winter, the number of natural freezing and thawing cycles for the entire vertical extent of the variable water level zone was established (Fig. 6-2). This research showed that concrete in the variable water level zone, during a year of average severity with 400 possible freezing and thawing cycles caused by changing temperatures, actually experiences only 240 natural cycles.

The concrete samples being tested by the All-Union Scientific Research Institute of Transportation Construction accelerated method for frost resistance at the Murmansk Weather Station, showed no deterioration whatsoever after eight years, during which they underwent 5,000 accelerated /140/ cycles of freezing and thawing, which is equivalent to 6,200 standard cycles.

In severe winters (with persistent temperatures below -18°C , -20°C), in the zone of the structure between the high water level of the mean-quad-rature and mean-syzygial tides, a half-meter layer of solid ice appears which forms thermal insulation for the concrete during the coldest period lasting one to one and one half months. No damage of any sort to the structure was detected from this icing.

In spring the surface of the concrete up to the mean level of many years begins to be intensively overgrown with a solid blanket of barnacles, hydroids and seaweed, which, on the whole, perish during the summer, but by winter a solid crust-like brittle layer remains on the concrete, with seaweed up to 5-7 centimeters long, and covered with a bacterial slime. The crust and slime are easily scraped off. No damage to the concrete was observed from this growth.

After three years in a sea environment, core samples 155 millimeters in diameter and height were drilled from various parts of the unprotected surface of the concrete from the zone above the water. Tests gave the following results: strength 700-857 kgf per cm^2 (design strength 400 kgf per cm^2); permeability to water, 10 kgf per cm^2 (V10); coefficient of permeability, 3×10^{-9} centimeters per second; water absorption from 3.3 to 3.9%; volume mass 2.51 grams per cm^3 ; volume porosity from 8.3 to 9.8%.

After erection of the TPP block, while it was still in the construction dock, all sections of dubious quality underwent ultrasonic testing after repair of the cavities. Sections of the TPP block which contained cracks (Section 9-2) were also tested ultrasonically. The tests showed that the cracks did not penetrate the entire thickness of the concrete, that they did not extend to the structural seams, and that the concrete was homogeneous.

The concrete in some of the sections was tested throughout the vertical extent of the block in the finished structure (including the construction joints of the individual members) for uniformity of mix in terms of strength and large aggregate content. These tests showed that the mix was homogeneous and that the uniformity of concreting by blocks in terms of strength was over 0.95 and in terms of large aggregate content it was 0.9-0.95, which confirms the correctness of the concreting techniques developed for the TPP. /141/

Testing the concrete of the TPP block for water tightness was carried out in several stages. Production control concrete samples were tested for water tightness and were rated higher than V8. The concrete of the underwater part of the TPP block was tested for water tightness by filling the spacings with water, while the above-water concrete was tested by hosing it with water at a pressure of 5-7 kgf per cm². This revealed leaks at only a few points in externally unnoticeable honeycombed sections and around a number of fastening dowels for holding the forms. These leaks were eliminated (section 9-2).

During flooding of the construction dock, and during towage of the TPP block to Kislaya Guba, careful monitoring of all spacings showed no seepage or even sweating of the concrete. When the suction pipes of

the TPP were filled at Kislaya Guba a few leaks in the block were found and drained. These leaks appeared because two previously unnoticed honeycombed sections had not been sealed in the places where the regulating and monitoring apparatus had been embedded.

Chemical analysis of the water drained from one of these honeycombs showed that the cement of the TPP concrete is resistant to the action of corrosive sea water.

During testing for water tightness, and throughout the subsequent period of operation, the seams between the blocks and the construction joints between the precast members were found to be watertight. The concrete surface of the 15-centimeter walls was dry throughout the vertical extent of the chambers.

Resistance of the concrete of the TPP to mechanical attack was tested during the throughput of ice from Kislaya Guba through the sluice in the winter of 1968-1969. The unprotected concrete surface of the pier caps was not damaged after the passage of blocks of ice 5 x 7 meters in area and 20 centimeters thick.

Thus, a concrete was obtained for the Kislaya Guba TPP which satisfies the requirements called for in the construction of tidal electric power plants and sea structures operating in tidal seas.

6-2. Thermal Insulation and Waterproofing.

The concrete which was specially developed and used for the construction of the floating structure of the Kislaya Guba TPP, by virtue of its high frost resistance and watertightness, is unaffected by low freezing temperatures and has a high frost resistance. However, calculations and experimental research on the thin-walled design of the TPP structure have

/142/

shown that, in view of the significant difference in temperatures between the upper and lower (submerged) parts of the structure, significant (up to 40 kgf per cm²) normal tensile stresses arise which exceed permissible levels [46]. In addition to the tension in the structure resulting from the general effect of temperature, there are additional tensile stresses caused by the local effects of the fluctuating water level caused by the tides. Study of these effects for conditions at the Kislaya Guba TPP, carried out using V.S. Luk'yanov's hydraulic analogy method at the Central Scientific Research Institute of Structural Parts Laboratory, showed that such stresses could also attain significant values (27.6 kgf per cm²).

Of course, the simplest and most reliable method of doing away with the tensile stresses or significantly weakening them is to "clothe" the entire zone of variable water level and the above-water part of the structure with a thick "coat" of timber slabs and asphalt. Thus, to decrease the stresses to 10 kgf per cm² would require a 5-centimeter thick layer of wood slabs, while complete elimination of the stresses would require wood slabs 15 centimeters thick and 20-30-centimeters thick layer of a cinder-asphalt mixture. However, such a solution for the floating structure is unacceptable because the wood slabs under conditions at the TPP could be attacked by wood-borers in one year, the fastenings corrode quickly in sea water, and the weight of such insulation (up to 500 kilograms per m²) would have increased the draft of the block to such an extent as to prevent it from being towed out to sea from the construction dock.

For this reason, the research organizations (Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, All-Union Scientific Research Institute of Hydraulic-Engineering, Central Scientific Research Institute of Structural Parts, and All-Union Scientific Research Institute of Hard Alloys) were faced with the problem of creating thermal insulation for the flating block of the Kislaya Guba TPP which would be of low volume weight while possessing good thermal insulation properties and be durable under the given natural conditons.

As a result of research carried out in 1961-1965 the following coverings were proposed for the Kislaya Guba TPP:

/143/

1. A cast asphalt-claydite-concrete insulation: 36% asphalt with 35% clay filler and admixtures of mineral powder with rubber powder in a wooden 5-centimeter thick casing (All-Union Scientific Research Institute of Hydraulic Engineering).

2. A covering of 5-centimeter thick wooden slabs over a 10-millimeter thick layer of asphalt mastic (Central Scientific Research Institute of Structural Parts).

3. Epoxy resin foam thermal insulation composed of foamed plastic on an epoxy resin base (Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute) [39].

In 1964-1966 these coverings were applied to the surfaces of a cubic TPP fragment and reinforced concrete plates and tested at the Murmansk Weather Station of the Central Scientific Research Institute of Structural Parts.

The following results were obtained.

In the case of the first covering (All-Union Scientific Research Institute of Hydraulic Engineering) the insulation separated from the concrete after 200-360 accelerated cycles of freezing and thawing (testing

of this covering was carried out twice). The second covering (Central Scientific Research Institute of Structural Parts) showed no damage after 3,145 cycles. The durability of the third covering (Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute) varied depending on the structure and method of preparation of the epoxy resin foam.

Epoxy resin foams which were not reinforced with fiberglass began to scale off from the surface and cracks appeared in the coverings after 100-200 cycles (with admixtures - after 700 cycles). In the case of reinforced epoxy resin foam, after 3,000 cycles there was some scaling in the variable water level zone (it is supposed that hardening of the epoxy resin foams occurred at low temperatures).

The epoxy resin foam PEP-1 (Table 6-3), which had been kept during preparation in an enclosure with a stable temperature not lower than $+20^{\circ}\text{C}$, was not damaged after 3,000 accelerated cycles of freezing and thawing. The durability of coverings made of epoxy resins foam, as manifested during testing for frost resistance, showed their clear superiority over traditional insulating coverings (Table 6-2), which made it possible to choose them for the Kislaya Guba TPP.

Table 6-2 shows that epoxy resin foams have the lowest volume weight, possess high waterproofing and thermal insulation properties (a 2-centimeter thick layer of epoxy resin foam is equivalent to a 20-centimeter thick wood slab), when reinforced are sufficiently durable in terms of compression and flexure, are resistant to wear, adhere well to the surface of the concrete, do not have components which are corroded by sea water and, finally, the technology of their manufacture and application does not require special mechanisms. This insulation may be applied at temperatures as low as -5°C .

In order to obtain epoxy resin foam, hydrogen, containing polyethyl hydrosiloxane (GKZh-94), is passed through it. The hydrogen, interacting with the epoxy resin hardener PEP (polyethylene polyamine), evolves a large quantity of gas (ammonia) and simultaneously foams and hardens the material. Prior to its use at the Kislaya Guba TPP the epoxy resin foam insulation was tested over a small area during repair of the concrete at the Gorki Hydroelectric Power Plant and, after 1,200 cycles of freezing and thawing, showed no defects, while sections sheathed with three layers of fiberglass showed the effects of periodic abrasion by ice [37]. At the Kislaya Guba TPP, however, harsher climatic conditions awaited the epoxy resin foam covering than at the Gorki Hydroelectric Power Plant because at the TPP it had to be effective in the variable level zone of /145/ corrosive sea water and, in addition, this covering was, for the first time, to be installed under industrial conditions over a relatively large area. For this reason, in the summer of 1966, experimental operations were carried out at the construction platform of the Kislaya Guba TPP on the preparation of the epoxy resin foam and its application to the concrete surface of the floating TPP block being constructed.

The prepared PEP-1 mixture (Table 6-3) was poured into a 3-centimeter opening between the rigid backing of the form and the concrete surface of the block, which improved the adhesion of the epoxy mixture to the concrete. The form was made of a precast 2-centimeter plate composed of the very same epoxy resin foam, reinforced on both surfaces with a layer of fiberglass. The 0.7 x 0.9 meter plate was manufactured in a horizontal wooden base platform. Vertical forms complicated the manufacturing process and had to be rejected. The plates were manufactured at an air temperature of +15°C to -1°C. The main difficulty at this time was

Table 6-2

Comparative characteristics of insulating coverings, examined for
the Kislaya Guba TPP [38]

Indices	Insulating material			
	Wood	Asphalt mixture	Asphalt-claydite-concrete	Epoxy resin foam
Volume weight, kg/m^3	500-600	1,500-1,800	1,000	180-250
Ultimate compressive strength, kgf/cm^2	450-500	-	40-42	24-50
Ultimate tensile strength, kgf/cm^2	900-1,150	-	-	30-35
Ultimate flexure strength, kgf/cm^2	700-900	-	-	30-35
Coefficient of thermal conductivity $\text{kcal(mh}^\circ\text{C)}$	0.20-0.40	0.40-0.50	0.18-0.20	0.03-0.05
Coefficient of thermal diffusivity, m/h	10×10^{-1}	11×10^{-1}	$8-9 \times 10^{-4}$	$5-8 \times 10^{-4}$
Heat capacity $\text{kcal (kg}^\circ\text{C)}$	0.32-0.78	0.21-0.36	0.25	0.17-0.32
Thermostability, $^\circ\text{C}$	-	80-100	150	150-170
Coefficient of thermal linear expansion, deg^{-1}	$5-7 \times 10^{-6}$	-	11.0×10^{-6}	$20-36 \times 10^{-6}$
Water absorption in terms of concrete, %	16-18	3-6	0.45-1.0	4-6
Watertightness, kgf/cm^2	-	5	5	5
Adhesion to concrete	none	low	low	high
Frost resistance, cycles	6,000	-	100-300	More than 3,000

measuring out the components of the mixture, whose volume changed by up to 15% depending on the temperature. The hardening process in the form lasted 10-30 hours depending on the air temperature. Separation of the fiberglass was observed in plates removed more quickly from the forms. The compressive and tensile strength during flexure of 90-day old sample forms varied from 12 to 48 kgf per cm². Finished plates were glued with a PPEP-1 mixture (Table 6-3) and were fastened to the construction scaffolding for their period of hardening.

Fiberglass fabric (ASTT_b-C₁) was then glued to the plates with the aid of compound ZPEP-1 (Table 6-3), after which two more layers of this glue were applied with the addition of powdered aluminum serving as a light-reflecting coating.

The epoxy resin foam mixtures were prepared by means of a very simple method: the epoxy resin was heated in a water bath (metal tank) with the water heated to +60°C, plasticized with polyether MGF-9, mixed with the foaming agent (GKZH-94) and the hardener (PEP) while being stirred for three minutes, either by hand or with the aid of a simple stirrer fabricated at the site, until a uniform mixture was obtained. The hardener was added to the mixture immediately prior to its use. The temperature of the mixture during pouring was 35-42°C, depending on the air temperature. The components were measured out by weight.

/146/

The experimentally developed design of the thermal insulation and its manufacturing techniques were adopted for the TPP block. In late 1966- early 1967, a special epoxy resin installation was located side by side with the construction area of the TPP block, consisting of a building for the preparation of the compounds and the precast epoxy resin plates, a storehouse for the components and a storehouse for the finished products.

Table 6-3

Composition of the thermal insulation of the TPP

Component	Purpose of Component	Content of component in the mixture, parts by weight		
		PEP-1 (for pre-cast plates and casting)	PPEP-1 (for gluing plates to concrete)	ZPEP-1 (for gluing fiberglass fabric and light-reflecting covering)
Epoxy resin (ED-6)	Binder	100	100	100
Polyether (MGF-9)	Plasticizer	20	20	20
Polyethyl-hydrosiloxane (GKZh-94)	Foaming Agent	10	2.5	-
Polyethylene-polyamine (PEP)	Hardener	15	15	10
Acetone	Solvent	-	-	20
Powdered Aluminum	Pigment	-	-	10-30

All of the buildings were linked together and formed a single production line. In 1967, 2,300 epoxy resin plates were manufactured in horizontal forms. In the fall of 1967, at air temperatures of -5 to $+10^{\circ}\text{C}$, the thermal insulation was applied to the external walls of the block, between February and April of 1968, using winter enclosures and at temperatures of -5 to $+10^{\circ}\text{C}$, it was applied to the piers, protective beams, and the spillway walls (Fig. 6-3). In the summer of 1968, 5-centimeter thick epoxy resin plates were glued to the bottom of the spillway and the retaining walls of the TPP block chambers. A total of 1,500 meters² of thermal insulation was applied to the TPP¹.

The high cost of the insulation (72 rubles per m^2) is explained by the cost of the materials (90%), whose manufacture is still on an experimental basis. Their industrial production is envisaged in the next few years, at which time the cost of the components will drop by 8-10 times. Even by 1970 their cost had dropped by 30%. The actual labor costs comprised 0.733 man-shifts per one meter², including 0.48 for manufacture of the plates. This index can also be significantly lowered when manufacturing the plates under factory conditions. The cost of the equipment (compressor for cleaning the surface of the concrete, and the apparatus for mixing the compounds), comprised 0.34 rubles per m^2 .

/147/

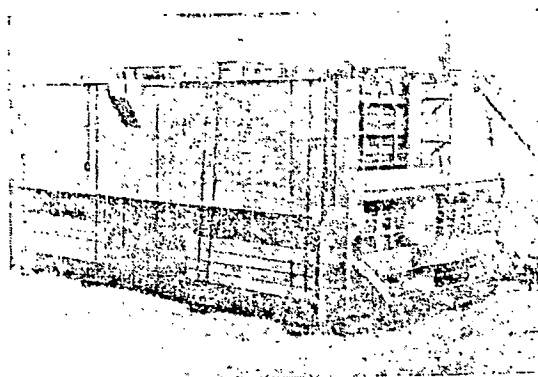


Рис. 6-3. Общий вид железобетонного наплавного блока ПЭС с защитными покрытиями. 18/VI 1968 г. фото А. С. Фирфарова.

В зоне полного погружения — легтезоксидная гидроизоляция (темный тон). В зоне переменного уровня и надводной зоне — пенозоксидная теплогидроизоляция (светлый тон).

Figure 6-3. General view of the TPP reinforced concrete floating block with protective coverings. 18 June 1968. Photographed by A.S. Firfarov.

Asphalt-epoxy resin waterproofing in the fully submerged zone (dark color). Epoxy resin foam thermal insulation in the variable level and above-water zone (light color).

¹The work of developing and applying the epoxy resin thermal insulation was carried out by the Engineering Departments of the TPP, the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute (V.I. Sakharov and V.A. Sirotkin) and the GRP (I.N. Usachev - application techniques).

Despite the high cost of the starting materials, the experience of using them at the Kislaya Guba TPP is showing that, even at the present time, epoxy resin coverings for structures erected by the floating method (with identical thermal insulation capacity) are cheaper than traditional coverings and require less labor input.

Resistance thermometers, installed in the concrete in the variable level zone and in the upper part of the structure, are used for experimental monitoring of the thermal effect of the insulation under field conditions (Fig. 5-6).

In addition, four resistance thermometers, number 762, 690, 792 and 756, were installed immediately beneath the thermal insulation at measurement points 1-1 and 3-3, at levels 39.3 and 41.3 meters (Fig. 5-6). Simultaneous monitoring of temperatures using these thermometers and thermometers number 139, 169, 147 and 177, located directly beneath them within the concrete, made it possible to study the distribution of temperatures in the system "external environment-thermal insulation-concrete". Observations showed that the diurnal temperature fluctuations of the surrounding air are almost completely damped, with the result that, as a rule, the temperature changes only by tenths of a degree. As for decreases in monthly and seasonal temperature changes, Figure 6-4 gives a good picture of the situation. Curve 1 shows the mean daily temperatures of the surrounding air at the outside wall of the pier from the side of turbine unit No. 1, curves 2 and 4 show the temperature at the contact between the thermal insulation and the concrete as detected by thermometers number 756 and 792, and curves 3 and 5 show the temperatures in the adjoining concrete as detected by thermometers 177 and 147. It may be seen from the curves that for a maximum outside temperature drop of 30°C there is a corresponding temperature drop beneath the thermal insulation of 6.5 degrees (central part of the variable level

zone, a drop of 5 degrees within the concrete (central part of the variable level zone), and 2 degrees (lower part of the variable level zone). Thus, a 15-fold decrease in the amplitude of temperature variations is achieved within the concrete and a 5- to 6-fold decrease at its surface.

In order to determine the thermophysical properties of the thermal insulation developed by the Scientific Research Institute for Constructional Physics, thermocouples and heating wires were installed on several of the epoxy resin foam plates while they were being glued; the detector wires were brought into the monitoring and automation control room. Testing revealed the stability of the coefficient of thermal conductivity in the underwater and above-water zones and its increase in the variable level zone. This was apparently caused by both mechanical damage and the manufacture of the insulation (two layers were separated by fiberglass fabric, which turned out to be a conductor of dampness). The strength of the insulation was also found to be inadequate when tested under more severe conditions than those at the Kislaya Guba TPP. /150/

When water was admitted from beneath the gate to the spillway section, which is 100 m^2 in area, the upper reinforcing fiberglass fabric layer was torn away. All of the damaged sections were repaired and no new damage was discovered.

In the summer of 1968 the concrete surfaces of the block in the constantly submerged zone, over a total area of $2,000 \text{ m}^2$, was waterproofed with an asphalt-epoxy resin compound developed by the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute.

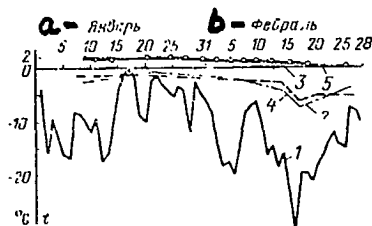


Рис. 6-4. Графики хода среднесуточных температур наружного воздуха, наружной грани бетона стенки бычка и внутри стенки.

1 — температура наружного воздуха; 2 — показания термометра сопротивления под теплоизоляцией на отметке 41,3 м (№ 756); 3 — то же в бетоне (№ 177); 4 — то же под изоляцией на отметке 39,3 м (№ 792); 5 — то же в бетоне (№ 147).

Figure 6-4. Curves of the mean diurnal temperature of the outside air, external face of the pier wall concrete and interior of the wall.

1 — temperature of outside air; 2 — resistance thermometer readings beneath the thermal insulation at the 41.3 meter mark (number 756); 3 — as above within the concrete (number 177); 4 — as above beneath the insulation at the 39.3 meter mark (number 792); 5 — as above within the concrete (number 147); a — January; b — February.

Composition of the waterproofing layer (in parts by weight):

Epoxy resin (ED-6)	100
Coal tar (KUS)	100
Acetone:	
for the base coat	100
for the main mixture	50
Polyethylenepolyamine (PEP)	10

The waterproofing was applied in several layers: one base coat and three coats of the main mixture. The waterproofing was applied to the concrete with the aid of a paint sprayer. Twenty parts by weight of aluminum powder were added to the third main mixture layer for the reflection of light.

Three coats of antifouling KhV-53 paint were applied over the asphalt-epoxy resin waterproofing on one side of the submerged part of the block. The durability of the asphalt-epoxy resin waterproofing, both reinforced and non-reinforced, was first tested for frost resistance by the accelerated method at the Murmansk Weather Station of the Central Scientific Research Institute of Structural Parts and showed no damage after 3,500 cycles of freezing and thawing.

The other side of the underwater part of the block was covered with fiber glass fabric (ASTT_b-C₁) instead of the KhV-53 antifouling paint. Underwater investigation, after a three-year period of operation, established that although the treated surface was overgrown with barnacles and seaweed, there was no damage.

/150/

In the summer of 1971, after three years of operation under the conditions at Kislaya Guba, it was established by underwater investigation that the asphalt-epoxy resin waterproofing (reinforced and with antifouling paint) was undamaged (except for 1.0-1.5 millimeter deep scratches sustained from falling rocks during construction of the connecting dams).

Waterproofing of the seams in the upper covering of the TPP block was also accomplished with reinforced (three layers of fiber glass fabric) asphalt-epoxy resin insulation. No seepage was observed through the waterproofed seams.

In summing up the experience of developing and applying the epoxy resin foam thermal insulation covering (volume weight 200 kilograms per m^3 , coefficient of thermal conductivity 0.04 kcal/per (mh°C), water absorption - 5% by volume), the effectiveness of the insulation under the conditions of the Kislaya Guba TPP is established. Should it be necessary to use the thermal insulation under more severe conditions (at the Mezen TPP, for example), further research would be required.

The cost of this insulation was found to be lower than the cost of traditional insulations made of wood and asphalt for structures erected by the floating method. Low labor costs in the industrial manufacture of epoxy resin form-sheathing and a possible 8-10-fold decrease in the cost of starting materials for mass production will make it possible to significantly decrease the cost of this insulation.

6-3. Hydrophobic Soil for Filling the Voids in the Floating TPP Structure

In order to ballast the structure in the zone of variable water level a hydrophobic soil, recommended by the Scientific Research Institute of Bridge Construction and the Central Scientific Research Institute of Structural Parts, was employed. The hydrophobic soil, a waterproof and water-repellent mixture of type 80 or 100 mazut with sand, was previously successfully used in ballasting the buttresses of bridges across the Oka River at Serpukhov and Kashira.

Research carried out at the Central Scientific Research Institute of Structural Parts [31], showed that the hydrophobic soil does not allow water to pass through it at pressures up to 2 kgf per cm^2 , does not accumulate moisture in the presence of a heat flow and contact with water, is not washed away by water flowing over its surface, and deforms easily at

temperatures below freezing under a load of 0.5-1.0 kgf per cm². The volume weight of the soil is 1.9-2.0 tons per m³.

/151/

The use of hydrophobic soil under the conditions of the northern tidal sea necessitated that additional research be conducted. In order to do this, at the Murmansk Weather Station of the Central Scientific Research Institute of Structural Parts, hydrophobic soil was used to fill an empty reinforced concrete fragment which was one of the samples for testing the frost resistance of the TPP concrete. During the testing period the fragment withstood 6,500 cycles of freezing and thawing (recalculation to standard cycles by the method of the Central Scientific Research Institute of Structural Parts). The hydrophobic soil did not lose its properties during these tests.

Experimental testing of the technology of preparation and installation of the hydrophobic soil under industrial conditions in the fall of 1966 with fragments of the TPP structure revealed that the hydrophobic soil (mixture of 320 kilograms of type 100 mazut and 1 m³ of sand) is readily selfpacking and that it may be placed in vertical and even horizontal voids in the structure.

The availability of mazut, however, under conditions at the Kislaya Guba TPP presented significant difficulties. Type 80 or 100 mazut from the Murmansk Heat and Electric Power Plant (to which it is delivered) may be obtained only at irregular intervals when this particular type is delivered. Transport (cisterns) had to be available, which had to be emptied into tanks, making it possible to reload them from trucks to barges for delivery to Kislaya Guba, to be unloaded onto the shore for subsequent preparation of the mixture with sand. In order to avoid these difficulties various measures were proposed, including proposals to replace all of the

hydrophobic soil with a mixture of ashes and cinders or to replace only the part of it within the variable level zone. These proposals had to be abandoned because the hydrophobic properties of an ash and cinder mixture are unreliable and do not last long under the given pressures.

The most suitable proposal was acknowledged to be the one according to which the hydrophobic soil would be placed only within the limits of the variable level zone between the constantly changing levels of the ebb and flow at quadrature (Fig. 6-5). This decreased the initially determined volume of the hydrophobic mixture by more than three times. At the upper levels it was decided to ballast the spacings with previous soils with drains for the water. The material for the previous layer was composed of screenings remaining at the Ura-Guba quarry after sorting of the soil /152/ for the foundation of the TPP.

In order to prepare the hydrophobic mixture, after inspection, the material from the Ura-Guba quarry was sifted for particles larger than 50 millimeters, and 170 m^3 of the hydrophobic soil was placed into all 39 of the spacings in the TPP structure between February and May of 1969. The long period of time required to carry out this work is explained by the above mentioned difficulties with the delivery of mazut. The hydrophobic soil mixture was prepared on an open platform near the TPP structure at air temperatures of $+2$ to -13°C and mixed for 3-5 minutes in a 250 liter capacity concrete mixer.

The mixture was prepared using type 100 mazut heated to $80-90^\circ\text{C}$ and sand-gravel soil heated to $20-40^\circ\text{C}$. The mixture was measured out by volume as follows: 320 kilograms of mazut per 1 m^3 of soil. the prepared mixture was loaded into a scoop which was brought to the block on an automatic dump truck and, with the aid of a crane on the TPP block, the mixture

The research and testing carried out at the Kislaya Guba TPP showed that, for the safety of structures operating in the area of variable water levels, the closed reinforced concrete voids may be ballasted with a hydrophobic soil composed of an impermeable pliable mixture of common sand and type 80 or 100 mazut. This mixture is distinguished by the stability of its properties over a long period of time under changing temperatures.

/153/

Inspection of this mixture when it was used to fill the void in the reinforced concrete fragment at the Murmansk Weather Station of the Central Scientific Research Institute of Structural Parts showed that it withstood 6,200 converted standard cycles of freezing and thawing without changing its properties, and it withstood over 1,000 such cycles in the TPP structure.

The work of preparing and placing the hydrophobic soil may be carried out in winter. A high mix quality may be achieved at such time.

The techniques employed when working with concrete are used in the preparation and placement of the hydrophobic soil.

The technical economic indices obtained during the use of the hydrophobic soil for ballasting are as follows:

productivity of the work of preparing and placing the mixture under field conditions in winter using a concrete mixer - 5 m^3 per shift;

labor input - 0.7 man-shifts per m^3 ;

cost of 1 m^3 of the hydrophobic soil at the site - 65 rubles (35% of this are the costs of transporting the components), which appears to be justified in view of the fact that filling the voids with concrete under conditions at the Kislaya Guba TPP would have been significantly more expensive.

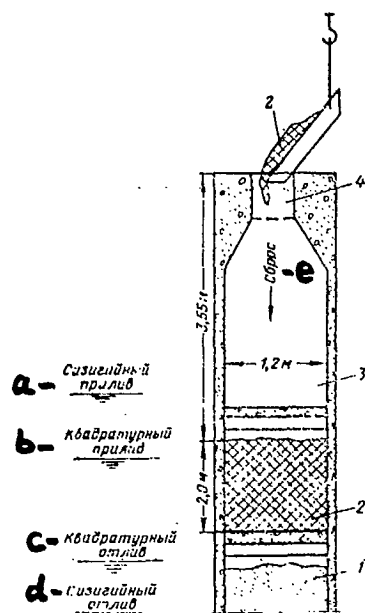


Рис. 6-5. Балластировка типовой шпанды Кислогубской ПЭС.

1 — песчано-гравийный карьерный грунт; 2 — гидрофобный грунт; 3 — щебень фракции 40-80 мм; 4 — бетон замощения шпанды.

Figure 6-5. Ballasting of a typical Kislaya Guba TPP spacing.

1 - sand-gravel quarry soil; 2 - hydrophobic soil; 3 - 40-80 millimeter crushed stone fractions; 4 - concrete for finishing the spacing; a - syzygial flood; b - quadrature flood; c - quadrature ebb; d - syzygial ebb; e - drop.

was dumped 5 meters into the spacing (Fig. 6-5). The mixture was effectively selfpacking at its dumping temperature of 25-30°C. The temperature regime for preparation and placement of the hydrophobic soil mixture was adjusted experimentally, right at the construction platform, in order to obtain a homogeneous mixture which would not separate into layers during placement.

Chapter 7

PROTECTION OF THE STRUCTURE AND EQUIPMENT FROM THE CORROSIVE ACTION OF SEA WATER

- Protection of the equipment and steel structures of the TPP from corrosion turned out to be significantly more difficult than for river hydroelectric power plants because at Kislaya Guba they are subjected to the corrosive action not of fresh water but of salt water with constant, periodic changes in levels and high current speeds. As a consequence of the high degree of biological fouling of the structures, the struggle to keep them from corroding must be combined with the prevention of fouling.

The constant change in the level of the sea water and its contact with the concrete over a large surface area leads to its saturation of the concrete as a result of capillary suction. For this reason, yet another problem had to be solved at the TPP - protection of the reinforcement metal of the reinforced concrete from corrosion. /154/

The need for cathodic protection of the hydraulic turbine unit was dictated by the fact that, at a sea water flow rate of 4 meters per second, standard steels and even certain types of alloy steels deteriorate at a rate of up to 0.5 millimeters per year, while at a water flow rate of 12 meters per second the deterioration rate is as high as 1 millimeter per year. Over several years this can result in the deterioration of the 10-centimeter thick facing of an unprotected bulb. For this reason, in order to ensure reliability of operation of the equipment and the structure, it was necessary to find effective measures for preventing their corrosion by means of protective coatings and electrochemical protection.

Cathodic protection of the turbine unit was provided for and partially carried out by the manufacturer of the unit, the Neyrpik Company (receptacles for the reference electrodes were built and anodes installed). Neyrpik, however, did not provide the rest of the protection components or the technical documentation for implementing it. The functioning of cathodic protection depends on the correct selection of its parameters. It was necessary to design protection applicable to the local conditions of Kislaya Guba as well as to handle automation of the equipment, keeping in mind the need for cathodic polarization of the unpainted and painted metal of the mechanical equipment and the turbine throughput duct. In addition, it was also necessary to propose reliable solutions for preventing deterioration of the mechanical equipment of the TPP, the sheet pile curtain, the embedded parts and other members.

7-1. Electrochemical Protection of the Metal Structures and Equipment from Corrosion.

Electrochemical protection is one of the most effective methods for preventing the corrosion of metals in the underwater zone. A special feature of it is its unlimited operation without having to carry out periodic underwater operations in order to, for example, restore coatings or regulate protective action.

Its principle of operation is cathodic polarization of the metal structure with direct current from an external source. The protective current from the anodes distributes itself unevenly over the structures being protected. Under the influence of this current the potentials of the cathodic sections of the structure change over to anodic values, which results in a decrease in the difference of potentials between the members of corrosion pairs and to the cessation or decreased intensity of deterioration of the polarized metal.

/155/

Two cathodic stations, working independently, are provided for the protection of the entire TPP complex. Each of them consists of a cathodic station, a control panel designed for regulating, distributing and controlling the currents in the anodic and draining lines, the anodes, the draining and anodic lines.

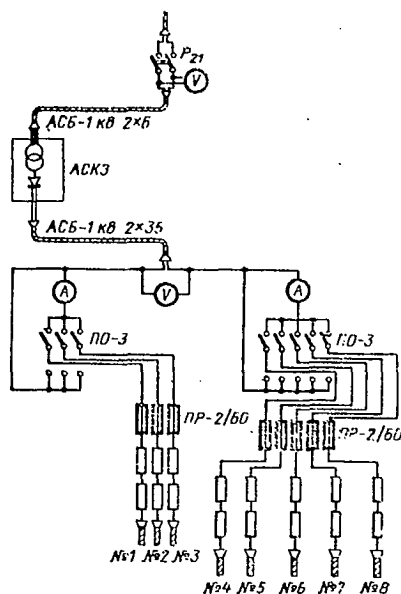


Рис. 7-1. Схема щита автоматической катодной станции для защиты проточной части турбины.

АСКЗ - автоматическая станция катодной защиты катодостатического типа; P_{21} - рубильник; ПО-3 - переключатели однополюсные; ПР-2/60 - предохранители; № 1-3 - отсосные линии; № 4-8 - анодные линии.

Figure 7-1. Schematic diagram of the control panel of the automatic cathodic station for protection of the throughput duct part of the turbine.

ACK3 - automatic cathodic protection station of the cathostatic type;
 P_{21} - knife switch; ПО-3 - single-pole switches; ПР-2/60 - cutouts;
 No. 1-3 - draining lines; No. 4-8 - anode lines.

The turbine's cathodic installation (Fig. 7-1) protects the cowling of the runner wheel chamber, the guiding apparatus, the surfaces of the bulb which are washed by sea water, and the blades of the runner wheel. The TPP block installation polarizes the reinforcement of the reinforced concrete TPP block, the sheet pile curtains of the dams and part of the turbine throughput duct. The cathodic protection works together with the paint and varnish coatings.

The gates of the turbine water passage, the bottom weir and surface spillway openings are all protected by the same paint and varnish coating. Protective current is fed to these steel structures by means of built-in magnesium protectors. As a result of the electrode potential differences in the protector-structure pairs, which attain 1 volt, a current arises. As the magnesium protectors dissolve they provide the required flow in the circuit which is necessary for protection of the metal structure.

/156/

The experience of operating the gates has shown that cathodic protection is expedient only for bottom gates which operate underwater either constantly or for long periods of time. This protection is ineffective for the spillway gates, which are out of water for significant periods of time, inasmuch as they are mainly subjected to atmospheric corrosion.

Protection of the metal surfaces of the flow-through part of the turbine is accomplished with the aid of anodes provided for by Neyrpik during manufacture of the turbine. The State Scientific Research and Planning Institute of Off-Shore Oil designed a complete system of protection for the turbine and control over its operation, using anodes provided by Neyrpik. Soviet reference electrodes were installed in the receptacles and their readings were used to evaluate the effectiveness of the protection.

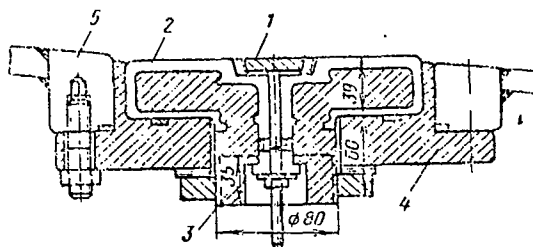


Рис. 7-2. Дисковый анод фирмы «Нейрпик», установленный на кожухе рабочего колеса и кожухе. 1 - титано-платинированный анод; 2 - пластмассовый изолятор анода; 3 - металлический сердечник анода; 4 - корпус анода; 5 - защищаемая конструкция.

Figure 7-2. Neyrpik disc anode installed in the casing of the runner wheel and the bulb.

1 - titano-platinized anode; 2 - plastic shield insulator; 3 - metal shield core; 4 - anode casing; 5 - structure being protected.

Thirty titano-platinized anodes were installed on the surfaces of the casing and the bulb over which water flows (Fig. 7-2). In order to facilitate control of the protection the anodes were combined into five independent groups of 10, 8 and 2 anodes per group (number of anodes in a group was determined depending on the surface to be protected). The current for each group is supplied by a separate anode line.

The draining lines of the cathodic protection are connected to various points on the protected sections of the turbine unit.

/157/

Control of operation of the protection of the turbine and casing of the runner wheel is accomplished in accordance with the design of the State Scientific Research and Planning Institute of Off-Shore Oil using nine copper-sulfate reference electrodes installed in special open receptacles built into the outer rim of the flow-through part of the turbine. In operation, however, as a result of their small size, the copper-sulfate reference electrodes quickly wear out because the high rates of flow during turbine operation result in rapid diffusion of the sulfate paste out of them.

The main parameter used to calculate the cathodic protection for the steel structures was the density of the cathodic current on the protected surface.

The design values of the cathodic current I were chosen taking into account the type of covering and on the basis of the area of the structure (Table 7-1) in terms of average values:

$$I = kjS, a,$$

where k - is the coefficient equal to 1, 2, which takes into account irregularities in the distribution of the protective current; j - is the protective current density, a/m^2 ; S - is the area of the surface being protected, m^2 .

The source of the current (Fig. 7-1) is a standard cathodic station KSS-3. The station consists of a switchboard with built-in regulating resistances, lines connecting the draining points with the positive pole of the current source, and the anode lines. In order to distribute the currents between the draining points and the anode groups, regulating resistances are introduced into each of these lines. The transformer is equipped with taps which may be switched to change the general voltage of the installation.

During the period of adjustment of the cathodic protection of the hydroturbine unit, the protection operating regimes were studied at various voltage levels of the source of currents for the protection. It was established that it is possible to obtain the design regimes by regulating only the rectifier taps of the KSS-3 (standard cathodic station). Six rectifier operating regimes were examined at currents of 6.5 amperes (first regime) to 34 amperes (6th regime).

Keeping in mind the good condition of the paint and varnish coating during the first period, with the turbine at a standstill it was possible to achieve the planned displacement of potential at only slightly more than half the planned current strength (27 amperes instead of the rated 44.4 amperes). /158/

Distribution of potentials over the protected structure was evaluated from the difference in potentials between the bodies and the corresponding reference electrodes. After start-up of the turbine the cathodic protection current was increased to 37 amperes and the potential on the surface of the flow-through section was increased to 1.1 volts. The potentials at various points are shown in Table 7-2.

In view of the variable operating regime of the turbine, and its stops during the periods of equalization of levels of the sea and basin, it was decided to automate the cathodic installation, which made it possible to significantly simplify operation of the protection as well as to save electric power. /159/

The use of cathodic protection over two years has shown that the planned regimes are easily carried out; quite even distribution of potential is achieved over the protected surface of the hydraulic turbine unit. Areas of damaged paint on the bulb of the turbine became covered with a dense cathodic residue through the action of the cathodic protection. The automatic control system independently chooses the system's operating regime in accordance with the condition of the applied coating.

Table 7-1

Design values of the cathodic current for the protected structures

Structure protected	Surface area protected	Type of coating (planned)	Accepted protective current density j , a/m^2	Rated cathodic protection current
Turbine unit (bulb and runner wheel chamber)	150*	Imported vinyl paint (cellobraz)	0.1	18.0
Runner wheel blades	21*	Coating not provided for in design (two blades painted with KhV-74+KhS-79 over a layer of VL-02	1.0	25.2
Spillway cover (water flows only over top part)	32*	KhV-53 paint	0.03	1.15
TOTAL	203	-	-	44.30
Gates:				
smooth surface	244**	Non-fouling KhS-79 or KhV-750 paint over four layers of KhS-720 on a VL-02 phosphate base coat	0.03	41.4
ribbed surface	900**			

*Cathodic protection

** Anodic protection

Table 7-2

Electrode potentials of the turbine under various operating regimes of the cathodic protection with reference to the built-in copper sulfate reference electrodes.

Potentials, v				
Reference electrode number ¹	Protection switched off (turbine not operating)	Protection on, current 17 amps (turbine operating)	Protection on, current 40 amps (turbine operating)	Protection on, current 40 amps, 3 hours after being switched on (turbine operating)
1	-0.70	-0.70	-0.85	-0.92
2	-0.70	-0.68	-0.72	-0.80
3	-0.73	-0.74	-0.76	-0.82
4	-0.74	-0.72	-0.69	-0.70
5	-0.72	-0.84	-1.07	-1.29
6	-0.65	-0.70	-0.86	-0.93
7	-0.695	-0.795	-0.90	-0.91
8	-0.69	-0.78	-0.80	-0.92
9	-0.70	-0.68	-0.84	-0.88

¹In each spacing (block well) three electrodes are installed.

In 1969, while the turbine was shut down for several months, the water was not completely pumped out from the turbine duct, as a consequence of which the lower anode was above the water level. Naturally, since the anodes were not in contact with the sea water the cathodic protection was inoperative. Upon inspection of the turbine, which had operated under this

regime for three months (cathodic protection in effect inoperative), corrosion was observed on the lower ribs of the turbine stator over an area of 1.4 m^2 . The lacquer and paint coating in these areas had completely deteriorated and the entire surface of the metal had become covered with the products of corrosion. In addition, deterioration was observed of the contact of the draining line with the anodes protecting the turbine in the collar connections of the turbine stator rim with the runner wheel chamber. This caused temporary disruption of the stator's cathodic protection system and resulted in its corrosion.

/160/

After the chamber was flooded and the cathodic protection was switched on all bare metal surfaces became covered with cathodic residues and the further spread of corrosion was stopped despite constant operation of the turbine and quite high rates of flow of the water. An inspection carried out in February 1972 revealed that all parts of the turbine, with the exception of the stator, were in good condition.

7-2. Electrochemical Protection of the Reinforcement of the TPP Structure from Corrosion

Particularly harsh corrosive conditions arise on the reinforcement of the reinforced concrete structures of the floating block of the tidal power plant. These differ significantly from the corrosive conditions at river hydroelectric power plants. In addition to the corrosive nature of the sea water, this is due to the small thickness of the protective layer of concrete over the reinforcement, which does not exceed two centimeters for 15-centimeter thick walls. The concrete is densely reinforced with two screens. For this reason, the steps taken to protect steel structures in fresh waters were acknowledged to be unacceptable for the protection of the reinforced concrete structure of the TPP block.

When cathodic protection is used at river hydroelectric power plants, in order to prevent the cathodic protection currents from penetrating adjacent reinforced concrete structures and to concentrate the currents in only the protected metal members of the mechanical equipment, the anodes of the cathodic protection systems is installed directly in the structural members being protected.

A different solution to this problem had to be found at the Kislaya Guba TPP inasmuch as it was necessary to protect not only the metal of the hydroturbine and the mechanical equipment but also the steel of the reinforced concrete block. This made it necessary to concentrate part of the current directly on the reinforcement. The metal parts of the mechanical equipment and the reinforcement of the reinforced concrete block has quite a large surface area and a separate powerful station was designed for their protection.

/161/

In designing the cathodic protection for the steel of the reinforced concrete, specific features were taken into account which complicate its realization:

possible flow of currents between the outer and inner reinforcing skeletons:

screening of the inner skeleton by the outer;

possible deterioration of the adhesion between the reinforcement and the concrete through the action of the hydrogen gas released at the surface of the metal during cathodic protection.

In order to prevent these negative effects and the possibility of stray currents arising between the outer and inner skeletons, all parts of the reinforcement were welded into an electrically single unit during construction.

Because screening of the inner skeleton by the outer reinforcing mesh complicated the establishment of protection from the common cathodic installation, protection was implemented using different anodes. The outer skeleton was protected by current from outer anodes while the inner skeleton was protected by current from anodes installed within the spacings.

In order to exclude the possibility of decreased adhesion between the concrete and the reinforcement the magnitude of the potential was strictly limited.

The cathodic station, located in the center of the TPP block (Fig. 3-1, position 24), handles the protection of the reinforcement of the reinforced concrete TPP block members, the sheet pile curtains of the connecting dams, and the embedded parts, with the aid of two external anodes (Fig. 3-1, position 18), installed on the sea and basin sides in the constantly submerged zone (i.e. below the low-water level). The anodes are in the form of two steel pipes, 43 meters long, 377 meters in diameter and with 11-millimeter thick walls. Their dimensions were chosen based on the need for the anodes to be in service for ten years.

Special internal pipe anodes were installed in the spacings which are filled with sea water and sand-gravel soil saturated with sea water.

The layout of the cathodic protection station for the block is similar to the layout of the cathodic protection station for the turbine unit (Fig. 7-1) with the single exception that there are ten anode lines. One of the four draining lines is connected to the embedded parts of the TPP, two are connected to the sheet pile curtains, one to the steel of the reinforced concrete.

To protect the large surface area of the block reinforcement a VAS-600/300 rectifier was chosen with a nominal current of 300/600 amperes and a voltage in the rectified current circuits of 24/12 volts. The anode lines (anodes and connecting lines) are the main elements determining the proper distribution of the protecting current. Installed in each anode line circuit is a cutout to protect it from short circuits and a group of resistors for regulating the current.

In developing the cathode station circuit, much attention was devoted to control of the operating regime of the protection and to measurement of the protecting currents in the anode and draining lines.

Special measures for current distribution control were taken for the steel of the reinforced concrete of the TPP block in connection with the possible danger for the reinforcement if it were overprotected.

At various levels in each spacing, three insulated steel reference electrodes were installed and their potentials were used to regulate the power of the current in the individual anode lines. The wires from each of the steel reference electrodes were led out to a switch on the monitoring board installed in the cathodic station building. The potentials were measured with the aid of a vacuum-tube voltmeter.

The protective layer of concrete (2 centimeters) over the reinforcement increases the uniformity of distribution of the potential over the surface of the reinforcing skeleton. In places where uninsulated steel parts come to the surface of the reinforced concrete the potential, at the moment the protection was turned on, was lower than over the rest of the surface. Before protection the average value of the potential over the block was 0.70 volts with respect to the copper-sulfate reference electrode. Twenty-four hours after the protection is switched on the potential reaches 1.0 volts.

It should be noted that the distribution of potential is uniform within the spacings both during operation of the protection and when it is switched off (absence of external cathodic polarization).

Analysis of the reinforcement potentials confirms the proper operation of the inner anodes, which testifies to the proper choice of the calculated protection currents (Table 7-3).

Table 7-3

Electrode potentials of the inner-loop steel of the reinforced concrete (in the inner spacings) with reference to the copper-sulfate electrode.

Potentials, v					
Electrode number	18 January 1969	21 January 1969	6 April 1969	9 December 1969	
	Cathodic protection operating regimes				
	Before being switched on	12 hours after being switched on	After 3 months of operation ¹	On	Off
1	-0.670	-0.870	-1.71	-0.81	-0.790
2	-0.490	-0.950	-1.34	-0.990	-0.810
3	-0.680	-0.832	-1.29	-0.780	-0.770
4	-0.675	-	-1.35	-0.910	-0.710
5	-0.665	-0.825	-1.30	-0.690	-0.690
6	-0.755	-0.854	-1.30	-0.940	-0.890
7	-0.680	-0.860	-1.27	-0.760	-0.730
8	-0.780	-	-1.34	-0.810	-0.770
9	-0.670	-0.900	-1.33	-0.890	-0.830
10	-0.07	-0.712	-1.21	-0.500	-0.500
11	-0.670	-0.958	-1.33	-0.810	-0.750
12	-0.670	-0.955	-1.34	-0.850	-0.800
13	-0.660	-0.816	-1.31	-0.880	-0.770
14	-0.680	-0.825	-1.05	-0.850	-0.800
15	-0.680	-0.700	-1.30	-0.530	-0.690
16	-0.680	-0.700	-1.30	-0.790	-0.770
17	-0.665	-0.971	-1.34	-1.035	-0.935
18	-0.665	-0.970	-1.32	-0.950	-0.860

¹ Cathodic protection switched to periodic (every other day) operating regime. Polarization current decreased from 120 to 95 amperes.

In order to limit the electrode potentials as they increased with time it became necessary to decrease the overall protection current from 165 (computed) to 95 amperes (actual), i.e. by 40%. But even at the decreased current (95 amperes) a rapid increase of potentials was observed. At this time they attained inadmissible levels during the course of one day (= 1.2 to -1.3 volts) at the outer contours. This made it necessary to change the operating regime of the protection from constant polarization of the reinforcement to periodic (once per week for 24 hours). Experience indicated the possibility of significant disruption of the reliability of the protection if its regime were not regulated automatically. The constant influence of cathodic protection on the reinforcement results in an accumulation of alkaline cathodic products in the zone near the electrodes and to rapid displacement of the electrode potentials in the direction of negative values. The potentials rise so rapidly that, during the course of a few hours, they reach values at which the gaseous generation of hydrogen is possible at the metal-concrete boundary. For this reason, in addition to the above mentioned transition to periodic regime in order to increase the reliability of operation of the steel of the reinforced concrete, and in order to reduce costs, the State Scientific Research and Planning Institute of Off-Shore Oil together with the Department of Automation, Remote Control, and Monitoring of the Azerbaydzhan Institute of Petroleum and Chemistry, developed a system for automatic regulation of the cathodic protection by means of pulsed supply of the protection current within the limits of the stipulated potential values.

Analysis of the possible methods of automating the electrochemical protection revealed the possibility of two solutions to this problem.

Even though the generally accepted method, which involves stabilization of potentials, ensures reliable operation of the protection, it is not technically and economically efficient. The second, cathostatic, method consists in switching off the protection once it has reached an upper, previously stipulated limit and switching it back on when the potential drops to an established lower limit. The regulated current remains unchanged throughout the entire period during which the protection is on. Only the lengths of the on and off periods change: the shorter the on portion of the cycle the cheaper the protection inasmuch as the expenditure of current and anodes is proportional to its duration of operation.

Automatic regulation of the equipment was accomplished using a high-ohm attachment. The regulation consists in changing the duration of the on and off intervals of the cathodic station depending on the rate of increase in potential of the steel of the reinforced concrete in the presence of constant polarizing current density ($j = 0.04 \text{ amp/m}^2$). Control of this type is significantly simpler than in systems based on the principle of potential stabilization, and it makes it possible to simply wire in the attachment to an operating cathodic station. Even during the first months of use, the reliability of operation was substantiated, with precise control of potential increase and stable maintenance of the prescribed limits.

Servicing of the installation (cathostat) consists in replacement of a circular chart and monitoring of the contacts (periodic inspection and cleaning).

/165/

Installation of the automatic cathostat solved the following problems of protecting the steel of the reinforced concrete of the TPP block:

it simplified operation of the cathodic protection (with respect to regular measurement of potentials), and its periodic switching on and off;

it increases the reliability of control of the operation of the cathodic protection by recording the potential on a circular chart;

it decreased the expenditure of electric power by the cathodic protection of the steel of the reinforced concrete (by approximately 50%, according to data obtained during the first five months of operation).

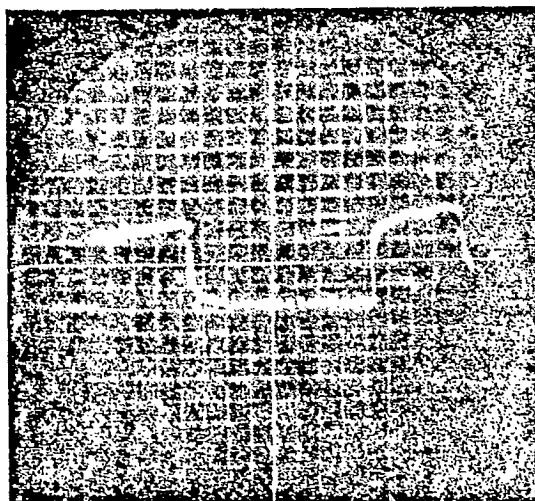


Рис. 7-3. Осциллограмма изменения потенциала арматуры железобетона в течение цикла включения, отключения катодата в натурных условиях.

Figure 7-3. Oscillogram of change in the potential of the steel of the reinforced concrete during an on-off cycle of the cathostat under field conditions.

Figure 7-3 shows an oscillogram of the change in potential of the steel of the reinforced concrete during one cycle. As may be seen, at the moment the protection is switched on the measured potential instantaneously increases to the value of the ohmic potential drop in the concrete. During the rest of the time the potential gradually increases due to polarization of the steel. At the instant it reaches its upper limit the protection is switched off and the potential instantaneously drops as a result of the disappearance of the ohmic potential drop in the concrete. During the rest of the time the potential decreases due to depolarization of the steel. When the lower limit is reached the protection is switched on again, etc. /166/

As may be seen from this oscillogram, the on and off periods may be quite short (each time marker point is 0.2 seconds) when the assigned upper and lower potential limits are close to each other. As the limits become narrower the ratio of on time to the total duration of the cycle becomes smaller and the cost effectiveness of the system increases. Short-duration switching of the protection, however, is inadmissible for instruments under contact control. Electromagnetic relays and starters have a limited service life and may be switched on only a finite number of times. If they are switched on too often they quickly break down and for this reason a contactless control system is required to obtain the most economic cathostatic regulator operating regimes. Inasmuch as control of the cathodic protection of the TPP reinforcement was handled using magnetic starters, it was decided to accept a non-optimal automatic control regime with relatively long on and off intervals: the on and off intervals were assigned by specifying upper and lower limiting values for the electrode potentials. They were taken to be -0.85 to -1.1 volts with respect to the copper-sulfate reference electrode.

Switching off the supply of current was accomplished with the aid of a high-ohm cathostat with an input resistance of the measuring circuit equal to one hundred megohms. Such a high resistance was necessary to ensure the stable operation of the copper-sulfate reference electrode. Switching was accomplished with the aid of a magnetic starter installed on the station's control panel.

The instrument is equipped with a mechanism for recording the potential, which made it possible to fully control the entire operation of protection. The automatic controls precisely maintain the limiting values of the electrode potentials. Stability of operation of the regimes of protection is ensured by the constancy of the potential of the reference electrode at the instant the protection is switched on, which testifies to the correct choice of the internal resistance of the measuring circuit.

/167/

Periodic changes in the operation of the regimes of protection are determined by the tides and the operating regime of the TPP hydroturbine. A drop in water level results in a decrease in protected surface area, an increase in the density of the polarizing current and, consequently, to a decrease in cycle length. When the level of the sea rises there is an increase in the polarized surface area, new unpolarized sections, which are not subjected to cathodic protection during ebb, come in contact with the water. Consequently there is an increase in on time and in the duration of cycles.

Examination of the protection operation graph shows that automation of the cathode installations, using a cathostatic regulator, not only increases the reliability of the protection and eliminates dangerous overloads of the electrode potentials at the metal-concrete boundary but it also decreases the consumption of the anodes and of electric power by more than three times.

This first Soviet experience in realizing cathostatic regulation of electrochemical installations is of significant interest not only as it applies to the tidal power plant, for it will also make it possible to examine the question of extending automation of electrochemical protection to mechanical and hydraulic power equipment at hydraulic power plants working in fresh waters.

Thus, the utilized electrochemical protection made it possible to fully protect the members of the flow-through part of the TPP turbine from corrosive deterioration through the action of sea water. The use of non-automatic cathodic protection cannot be recommended for the reinforcement of reinforced concrete due to the spontaneous rapid increase in electrode potentials, in the direction of negative values, until they reach potentials at which hydrogen is generated. Cathostatic regulation of cathodic stations makes it possible to simultaneously ensure reliable operation and increase its efficiency.

7-3. Anti-Fouling Coatings

Biological fouling, which is a dangerous enemy of shipping, was an object of close attention during the design of the TPP. After several years of sailing, a ship is placed in a dock for maintenance, fouling organisms are removed from its hull and screw, and its anti-fouling paint is renewed. However, the underwater parts of the TPP structure cannot be painted, and the turbine water ducts and the flow-through part of the turbine unit can become so encrusted during the course of a few years that normal TPP operation would be disrupted. For this reason, the extensive program of research included work on encrustation by microorganisms and seaweed at Kislaya Guba and also the development of protective measures against them.

/168/

To handle the first task, a team of scuba divers was organized and special institutes were involved (Institute of Oceanology of the Academy of Sciences, the North Polar Scientific Research Institute of Sea Fisheries, Moscow Institute of Marine Biology), who carried out careful underwater exploration not only of Kislaya Guba but also of possible construction sites of powerful TPPs in bays of the White Sea.

As a result of this work it was established [32] that the biomass of encrusting marine organisms (mainly barnacles) at Kislaya Guba may be as high as 5-6 kilograms per m^2 per year, which could result in serious difficulties in the operation of the TPP. In order to provide protection against fouling of the structure and turbine units of the TPP it was necessary to utilize the available experience of Soviet and foreign shipbuilding as well as corresponding work carried out at the Rance TPP.

Since the solution to this problem was of real importance to the further improvement of anti-fouling coatings for ships, hydraulic power structures and other applications, the Commission on Marine Fouling of the Soviet on Problems of Ocean Resources became involved, together with the All-Union Scientific, Engineering and Technical Society of Shipbuilding, which charged the Leningrad Affiliate of the State Research and Planning Institute of the Varnish and Paint Industry to carry out research in three areas:

- 1) examination of the effectiveness of anti-fouling paints, produced at the present time by the varnish and paint industry, under TPP conditions;
- 2) development of new special paints for TPP structures;
- 3) comparison of Soviet and French paints (proposed for the structures of the Rance TPP).

In 1964, field testing of possible coatings for the Kislaya Guba TPP was organized at the Northern Corrosion Station of the Institute of Physical Chemistry of the Academy of Sciences, USSR, at Dal'nii-Zelentsy¹, where conditions were almost the same as at Kislaya Guba. /169/

In 1967, floating and stationary testing units were built right at Kislaya Guba.

The stationary testing unit is located on the strand and is in the form of a crib in whose front part are two guide beams for installing plate magazines. The samples arranged on the testing unit are located in the average water level zone. This was chosen in order for the samples to be in water and in air for equal lengths of time. On the floating testing unit the coating samples are constantly submerged. On the basis of these testing unit and special laboratory tests, conducted by the State Research and Planning Institute for Varnish and Paint Coatings, under its author's² supervision a vinyl coating scheme which had proven itself in shipbuilding was recommended and implemented. The use of epoxy compounds, which also have high protective qualities, was deemed to be difficult under the given circumstances (since the painting operations are carried out at temperatures above +5°C). The following compounds were recommended and used for various types of protected surfaces.

1. In order to protect from fouling the asphalt-epoxy resin waterproofing of the TPP block, which is used completely under water, two layers of KhV-53 paint and one layer of KhF-751 paint were applied. This coating does not become encrusted for 4-7 years depending on the operating conditions.

¹This work was carried out for some time by B.V. Shirokolobov, Senior Research Officer.

²The work was carried out by E.S. Gurevich, Candidate of Technical Sciences and Yu.B. Shleomenzon, Engineer.

2. It was proposed that the thermal insulation and waterproofing of the variable level zone of the TPP be painted with two coats of KhV-750 (green) paint or KhS-79 (blue) paint. Coatings with these paints as a base are highly weather resistant and, under the conditions at Kislaya Guba, make it possible to protect the surface from encrustation for 2-3 years. Since, according to experimental data, fouling organisms do not damage the fiberglass fabric which is used to reinforce the thermal insulation, only experimental sections of the spillway were painted with KhV-750 and KhS-79 paints.

3. The metal surfaces which operate under conditions of anodic protection (the gates), are coated as follows: a phosphatizing base coat VL-02 - one layer, KhS-720al anti-corrosive paint - four layers, KhS-79 (blue) or KhV-750 (green) anti-fouling paint - two layers. A coating of this sort can withstand the alternating effects of sea water and atmosphere at low temperatures for 2-4 years. /170/

4. Under conditions of hydro-erosion, in order to protect from corrosion parts which are subjected to cavitation (for example, the blades of the runner wheel), they are usually manufactured out of high-alloy steel or covered with a high-alloy steel lining. An experiment carried out by the State Scientific Research and Planning Institute of Off-Shore Oil on one of the turbines of the Lenin Hydroelectric Power Plant on the Volga River revealed that this problem is effectively solved with the aid of cathodic protection. In addition, for minor cavitation effects (during pumping regime), the Leningrad Affiliate of the State Research and Planning Institute for Varnish and Paint Coatings studied the physico-mechanical and protective properties of coatings, their resistance under the conditions of a cavitating stream at speeds of 30 meters per second and their resistance

to jet impact at a speed of 25 meters per second. On the basis of the results of special and test-bench experiments it was recommended to coat the vanes of the guiding apparatus and the blades of the runner-wheel with: one layer of VL-02 base, four layers of KhV-74 paint and, to protect surfaces which are to be submerged for long periods of time in stationary sea water, two coats of KhS-79 (blue) anti-fouling paint. This coating was applied to ten vanes of the guiding apparatus and to two blades of the runner wheel.

Monitoring of the condition of the coatings during the course of two years of TPP operation and the preceeding three months which the structure spent in the open air (in the construction dock at Cape Prityka) and four months in sea water (during towing and after submergence at the site) showed that these coatings provided protection for all structures from fouling, which was also confirmed by the complete encrustation with barnacles, hydroids, and seaweed of the uncoated control sections. Monitoring of various structures revealed the following.

Detailed examination of the underwater part of the block, carried out in 1969, 1970 and 1971 by scuba divers, revealed that there was no encrustation on the anti-fouling coating of the asphalt-epoxy thermal insulation sections; no encrustation was observed over the thermal insulation which was coated with anti-fouling paints.

/171/

An inspection carried out in March 1970 showed that imported anti-fouling coatings (cellobraz) on the metal surfaces of the bulb, in combination with cathodic protection, perform well. Examination of these coatings in October 1970 confirmed this conclusion and revealed intensive formation of cathodic deposits around the anodes of the cathodic protection and at locations of insignificant damage to the imported coating. Several restricted sections (spots) were notices of blistering of the coating, which apparently

occurred as a consequence of exceeding the optimum cathodic protection current. The search for this optimum is, in fact, one of the main tasks of further experimentation especially in view of the fact that during the inspection of February 1971 blistering of the coating was observed over a larger area.

The coating of the spillway gates, after two years of an alternating atmospheric and seawater environment, was generally in good conditions but it showed local damage which was not completely covered with cathodic residues, which indicates the insufficient effectiveness of anodic protection in this instance.

The coating on the parts of the bottom gates which were under pressure was almost completely gone. In this area it is necessary, evidently, to increase the anodic protection.

It should be noted that the imported coating on the stator ribbing was found to be almost completely washed off and initial corrosion had appeared on the surface. This may be explained by the fact that the stator ribbing was manufactured out of carbon steel which could not withstand the corrosion which appeared during the time that the cathodic protection was switched off in the summer of 1969 when the turbine water conduit was not completely drained.

In connection with this, it is important to mention that the Soviet coating which was applied to 10 of the guiding apparatus vanes, as well as the imported coating (cellobraz) on the rest of the vanes was completely preserved, which testifies to the high quality of these coverings. The coating of the runner wheel blades on the sea side, after 2,000 hours of turbine operation, was almost completely washed off of one blade and deteriorated on the other (25%). From the basin side the coating was preserved to a significant extent (80-90%).

Thus, monitoring over a two-year period of the anti-corrosive and anti-fouling coatings showed their good durability and their protective qualities except against cavitation. The imported (French) six-layer coating of cellobraz with 6406 anti-fouling paint is in satisfactory condition and it is to be expected that its anti-corrosive and protective properties will last for another 2-4 years. It should be taken into account that this coating was applied in 1963 and had stood for five years in the atmosphere until being submerged. During laboratory testing in a 3% solution of NaCl it was established, however, that the rate of leaching out of the copper and the biological activity of 6406 anti-fouling paint are below those of the best Soviet anti-fouling paints. During test-stand testing of this coating under the conditions of the Barents Sea it showed initial encrustation after two years. Further observation of the condition of the TPP coatings will make it possible to obtain conclusive data about the service lives of the imported and Soviet coatings used there. A concomitant research task in this field at the Kislaya Guba TPP is that of obtaining the optimum cathodic protection regime, which would ensure longer service life of both the cathodic protection and the anti-corrosive and anti-fouling coatings.

In future, research will be carried out on new, more resistant and effective coatings which will be used at powerful TPPs and in ship-building (epoxy base paints, "black brass" - product of the cementation of copper by powdered zinc, paints with completely insoluble bonding agents with polyisobutylene). In addition, anti-corrosive coatings will be improved for protection of metal under conditions of hydro-erosion.

Chapter 8

SPECIAL FEATURES OF THE ORGANIZATION OF THE DESIGN, RESEARCH AND CONSTRUCTION WORK DURING ERECTION OF THE KISLAYA GUBA TPP

8-1. Research and Design Makeup and Organization

The construction of the Kislaya Guba TPP, using a totally new design and carried out by the floating method, introduced many problems which required a whole series of studies to solve. Numerous difficulties had to be overcome and requirements had to be met which were often conflicting. Indeed, the floating method of construction required that the block of the TPP structure have maximum buoyancy, i.e. lightness, making it possible to tow it by sea with minimal draft, and at the same time it required increased strength in order to withstand the onslaught of the sea. /173/

The design of this thin-walled structure, which is unusual for hydroelectric power plants, was complicated by the need to take into account the dimensional work of its members as well as the effect of dynamic loads and fluctuating tidal levels in the presence of general cooling of the upper part of the structure in winter and the positive temperature of its lower part, which is constantly under water.

A far from complete list of the problems which designers had to solve during construction of the Kislaya Guba TPP includes: absence during the initial period of design of industrially proven materials, and methods for protecting them, which could reliably withstand the corrosive effects of sea water; the unprecedented joining of the pressure structure with the foundation, carried out under water; the effects of ice, seaweed and marine fouling organisms; the use of a new bulb-type turbine unit and the creation of a new one with variable speed.

All of these problems found, if not a final solution, at least a sufficiently adequate one to make it possible to carry out the construction of the Kislaya Guba TPP as a prototype for powerful industrial installations. Clearly, this positive result of solving the above mentioned series of complex engineering problems shows the necessity of briefly describing the main points of research organization.

It is instructive, in this regard, that various institutes were involved in trying to find a solution to this problem and that they approached it from various directions. Understandably, it was possible to achieve this only within a framework of coordinated effort in this field (1963-1970) based on the significance of these tasks to the national economy.

Investigation of the natural conditions (Chapter 2) was carried out by the Marine Hydrology Group of the Leningrad All-Union Planning, Surveying and Scientific Research Institute of Oceanography, the Leningrad Branch of the State Institute of Oceanography, the Murmansk Directorate of the Hydrometeorological Service, and the Kola State Construction and Design Institute.

Computation of the structure, testing of it and creation of control and measuring apparatus (Chapter 5), were carried out jointly by the All-Union Planning, Surveying and Scientific Research Institute, the Central Scientific Research Institute of Structural Parts, the Central Scientific Research Institute of Communications, the Moscow Construction Engineering Institute, and the Rural Electrification Project.

The project of towing and decreasing the draft (Chapter 11) was carried out under the methodical direction of the Soviet for Reinforced Concrete Shipbuilding of the All-Union Planning, Surveying and Scientific Research Institute together with the State Planning, Design and Scientific Research Institute of Marine Transportation of the Ministry of the Maritime Fleet, the State Institute for Planning in River Transportation and the Leningrad Marine Planning Scientific Research Institute.

The underwater foundation project (installation and levelling of its gravel layer) was carried out by the All-Union Planning, Surveying and Scientific Research Institute and the All-Union Scientific Research Institute of Hydraulic Engineering.

Corrosion protection of the equipment and the structure (including the steel of the reinforced concrete) was carried out by the State Scientific Research and Planning Institute of Off-Shore Oil.

Protection from biological fouling (Chapter 7) was successfully solved with the aid of the Coordinating Commissions and Soviets in this field of the Scientific and Technical Society of Shipbuilding and the Academy of Sciences of the USSR with the involvement, upon their recommendation, of the Institute of Oceanology of the Academy of Sciences, the Institute of Physical Chemistry of the Academy of Sciences, and the State Research and Planning Institute for Varnish and Paint Coatings of the Ministry of the Chemical Industry.

Key to Figure 8-1.

1 - Ministry of Power Engineering and Electrification USSR, Scientific and Technical Soviet; 2 - State Committee of the Soviet of Ministers, USSR for Science and Technology; 3 - General Designer (All-Union Planning, Surveying and Scientific Research Institute); 4 - Client (Main Administration of Capital Construction, Management of Hydroelectric Power Plants being built on the Kola Peninsula, Kola Regional Administration of Power Systems Management); 5 - General Contractor (Office for Northern Hydraulic Engineering Construction); 6 - Chief Project and Construction Engineer; 7 - State All-Union Trust of Specialized Operations of the Main Construction Administration; 8 - General Repair Shop; 9 - Auto Transportation Office; 10 - Planning and Scientific Research Organizations; 11 - Subdivisions of the Main Hydraulic Construction Administration and Subcontracting Organizations; 12 - Research on Turbine Unit and Power Engineering, All-Union Scientific Research, Design and Technological Institute of Hydraulic Machinery; 13 - Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, All-Union Scientific Research Institute of Electric Power Engineering, Moscow Power Engineering Institute; 14 - Study of natural conditions, Expedition no. 3 of the All-Union Planning, Surveying and Scientific Research Institute, the State Institute of Oceanography, the Leningrad Oblast State Institute of Oceanography, the Institute Oceanology of the Academy of Sciences, the Moscow Institute of Marine Biology, the Northern Polar Scientific Research Institute of Sea Fisheries and Oceanography, the Institute of Physical Chemistry of the Academy of Sciences, the Northern and Murmansk Administration of the Hydrometeorological Service, the Kola State Administration of Planning for Construction; 15 - Research on building materials and their protection, the All-Union Scientific Research Institute of Transportation Construction, the All-Union Scientific Research Institute of Construction Materials, the Scientific Research Institute for Concrete and Reinforced Concrete, the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute; the All-Union Scientific Research Institute of Hydraulic Engineering; the Moscow Highway Institute; the State Research and Planning Institute of Varnish and Paint Coatings; the Scientific and Technical Society of Shipbuilding; the State Scientific Research and Planning Institute of Off-Shore Oil; the Reinforced Concrete Shipbuilding Design Office in Kherson; the Scientific Research Institute for Constructional Physics;

Key to Figure 8-1 Cont'd from page 206.

15 - Research and development of the structure, the Moscow Construction Engineering Institute; the Central Scientific Research Institute of Structural Parts; the All-Union Scientific Research Institute of Communications; the All-Union Scientific Research Institute of Hydraulic Engineering (permeability and hydraulics of the piers, stability of the slopes); the Scientific Research Institute of the Marine Planning; the State Institute for Planning in River Transportation; the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute; the Rural Power Engineering Projects; the Kola State Construction Project (electric power transmission line); Planning and Design Office for Steel used in Hydraulic Construction; 16 - Specialized Departments of the All-Union Planning, Surveying and Scientific Research Institute, Hydrometeorological Observatory; Communications; Hydroturbine Service; Water Reservoirs; Subsidiary Establishments; Architectural; Architectural Planning; Electrical Engineering; 17 - the TPP Department, Chief Project Engineer, Organization of Operations and Head of the Advanced Group; 18 - TPP Section; 19 - Construction Platform at Cape Prityka; 20 - Construction Platform at Kislaya Guba; 21 - Communications - Construction and Installation Administration; 22 - Gap in the construction dock - Arkhangel'sk Administration of Sea Routes; 23 - Blasting Operations - Trust for Drilling and Blasting Operations of the Main Administration for Special Types of Construction and Installation, State All-Union Trust for the Reinforcement of Foundations and Structures of the Main Administration for the Construction and Installation of Hydroelectric Power Plants; 24 - Excavation of the underwater foundation area, laying of the foundation and placement of the piers, - the Murmansk Marine Shipping Administration; 25 - Towage of the block - Merchant Marine Organizations; 26 - Draining of dock - All-Union Mine Draining Administration; 27 - Drainage System - Murmansk Ship Repair Yard of the Ministry of the Maritime Fleet; 28 - Sheet Pile - the Murmansk Marine Construction Administration; 29 - Roads at the power plant - the Murmansk Road-Building Administration; 30 - Installation Operations - State Electric Installation Administration, Local Electric Installation Administration, Leningrad Metal Plant; Electric Transmission Lines - Mechanized Column No. 27.

The creation of a very highly frost resistant concrete and its protection (Chapter 6) was carried out under the aegis of the Soviet for the Corrosion Protection of Concrete of the State Committee for Construction with the involvement of institutes and organizations working in this field (Scientific Research Institute of Concrete and Reinforced Concrete, All-Union Scientific Research Institute of Transportation Construction, Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, All-Union Scientific Research Institute of Construction Materials, All-Union Scientific Research Institute of Hydraulic Engineering, Moscow Highway Institute, Construction and Technological Offices of Reinforced Shipbuilding in Kherson and Nikolaev).

Creation of the hydrothermal insulation coatings and waterproofing materials (Chapter 6) was handled by the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, the All-Union Scientific Research Institute of Hydraulic Engineering, the All-Union Scientific Research Institute of Construction Materials, the All-Union Scientific Research Institute of Transportation Construction, and the Scientific Research Institute of Constructional Physics.

The power engineering research and work on developing a variable speed turbine unit (Chapter 4) were carried out by the All-Union Planning, Surveying and Scientific Research Institute with the involvement of the All-Union Scientific Research Institute of Electric Power Engineering, the Moscow Power Engineering Institute and the All-Union Scientific Research, Design and Technological Institute of Hydraulic Machinery.

In examining the work, described in this book, of creating effective hydrothermal insulation for the concrete, corrosion protection means, installation of the underwater foundation, etc, we see that it was precisely this method of utilizing the broad knowledge of all organizations, the creative participation of many organizations in the experiment, and the utilization of the chosen methods, which ensured success. Thus, it is clear that the positive results which were achieved were not the result of random chance but rather the result of systematic research involving the scientific resources of various institutes. The general makeup and organization of this research is given in the diagram pictured in Figure 8-1.

8-2. Special Features of the Organization of Construction of the Experimental Installation

Construction of the Kislaya Guba Experimental TPP was entrusted in 1962 to the Directorate (Trust) of the Northern Administration for the Construction and Installation of Hydroelectric Power Plants - a large industrial organization of the Ministry of Power Engineering of the USSR, which had previously erected a series of hydroelectric power plants on the Kovda River and had then begun construction of two powerful hydroelectric power plants on the Voron'ya River. The method decided upon was deemed to be well founded and appropriate for the floating method, which is based on the repeated utilization of an already existing production base of a large industrial construction organization. In doing this, it was not taken into account, however, that the erection of a relatively small-scale experimental structure, not in keeping with the main activities of the Construction Administration, would give rise to serious organizational difficulties. It appears that the incompatibility of the task of searching for the optimum solutions in the construction of the unusual thin-walled

structure, which is essentially research work, was in fact the real reason for the difficulties which cropped up even during the first period of construction. We will describe the circumstances of moving the construction dock inshore (section 2-3), construction of the embankment in Kislaya Guba in accordance with the regulations for damming rivers (section 10-1,b) instead of the method proposed in the project of utilizing the specifics of tidal flow (section 3-3), the insufficient number of workers and deviations from the technical conditions of the project for erecting the reinforced concrete floating framework (section 9-2) and, finally, delays in carrying out experimental explosions for loosening the soil in the underwater foundation area, which resulted in the proposal by the Northern Administration for Hydraulic Engineering Construction to build cofferdams and to erect the TPP structure in a drained area; a proposal which would actually negate the merit of the entire experiment and the need for carrying it out (section 10-1). These and other circumstances led all of the participants in the experiment to think that its implementation should be managed by the person who conceived (planned) it. Thus, a decision came down from the Ministry of Electric Power, USSR, stating that "in order to ensure the best conditions for realizing the new technical solutions incorporated in the project, the Chief Project Engineer was entrusted with the responsibilities of the Chief Construction Engineer of the Kislaya Guba Experimental TPP under the Northern Administration for Hydraulic Engineering Construction", and it was proposed that he "provide the TPP construction organization with the required material resources, technical personnel, and manpower". It is important to mention that by this decision "the responsibility for carrying out all technical decisions in the construction of the TPP was conferred upon its Chief Project and Construction Engineer".

/176/

This new organization, which was fully implemented in August 1967 by G.A. Khukhlaev, the head of the Northern Administration for Hydraulic Engineering Construction, and which united the direction of the project, the research, and the construction, had an immediate beneficial effect on the work. The efforts of the builders, designers and researchers were directed towards solving the most difficult problems, which had a positive effect on the successful surmounting of the problems. Thus, specialists from the Department of Polymeric Materials of the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, right on the construction platform and together with the construction workers and designers, used the most modern methods for filling ultrasonically discovered cavities and a few imperceptible cracks, which were reliably fixed and injected with epoxy compounds. The one-man management and the purposefulness of all participants in the experiment was especially clearly manifested during the most critical moments of construction, in emergency situations, when the struggle with the elements required precision and timing of all activities to the minute and even to the second [(for example, during movement of the block from the flooded construction area, its negotiation through the gap, its towage, its delivery to and submergence at the site, ballasting (Chap. 11) and, finally, operation of the crane during dismantlement of the embankment (section 10-1)]. Understandably, all this sharply accelerated the pace of operations.

In constructing this small experimental installation by the floating method, not only the equipment and experts of the Northern Administration for Hydraulic Engineering Construction were involved, but also those of such large organizations as the Rescue and Salvage, Shipraising and Underwater-Technical Operations Organization and the Murmansk Shipbuilding Administration. The best specialists from these organizations worked on

the construction of the Kislaya Guba TPP: Section Chief I.I. Veselov, Senior Foreman E.I. Makarov, Brigade Foremen N.A. Orlov, G.R. Smirnov, Excavator Operator Yu.Z. Kotkevich, Divers L.I. Larchenko, N.I. Shulichenko, B.F. Makhnev, and others.

Management unification also resulted in significant advantages in the selection of subcontracting organizations and relations with them. The large number of these organizations (Fig. 8-1) and the specific nature of the tasks performed by each were dictated by the innovative nature and the complexity of the solutions for this project, and urgently and intrinsically demanded a unified approach by the general designer and general contractor, especially in handling unforeseen situations such as, for example, the difficulties in draining the foundation area of the construction dock and in carrying out the unexpected requirements of the Arkhangel'sk Administration of Sea Routes for moving the TPP block prior to beginning the work of excavating the gap (section 11-2) or in solving the problems of bringing the block in and out (section 11-3).

In generally evaluating the positive experience of unifying the management of construction and design it should be noted that the results could have been even more effective if the Chief Construction Engineer had been invested with powers with respect to the State All-Union Trust of Specialized Operations, a subdivision of the Northern Administration for Hydraulic Engineering Construction, and if the personnel under his jurisdiction had not been under double leadership: to him as well as to the Administration of the State All-Union Trust of Specialized Operations (Fig. 8-1).

/177/

Finally, it should be concluded that unification of project and construction management, which was carried out as an experiment during construction of the Kislaya Guba TPP, could give significant results in ordinary as well as experimental construction.

8-3. Organization of Operations at the Two Sites.

Author's Supervision

One of the important features of the floating method of hydroelectric power plant construction, which is of crucial positive significance, is the possibility of simultaneously carrying out work on preparation of the foundation area and construction of the power plant structure. Thus, it is possible to reduce to a minimum the unavoidable interruptions in the construction of a hydroelectric power plant structure after preparation of the foundation area which, under the traditional method of power plant construction, results in extended periods of one to several years. Naturally, with the floating method, prior to beginning the construction of the power plant structure it is necessary to build a construction dock. With normal organization of operations, however, erection of the construction dock on the shore requires significantly smaller outlays of time and effort than removing the water from a large foundation area in a river bed (or in the mouth of an inlet of the sea).

In the case of this project, unfortunately, due to the organizational reasons stated above, it was not possible to fully realize this advantage but valuable experience was gained nevertheless, which proves the possibility of realizing the advantages inherent in the floating method of TPP construction when it is carried out on a large scale.

As was already noted above, two sites were chosen for the construction of the Kislaya Guba TPP: one at Cape Prityka for the construction dock, the second in the throat of Kislaya Guba - the TPP site. Let us examine the organization of work at each of these sites.

At the construction dock site, located between the waterline of Kola Bay and the Murmansk to Pechenga Highway, were located all of the necessary equipment and material for the erection of the floating TPP block, as shown on Figure 8-2. Also located in this area was the building for inspection of the imported equipment (6 x 11 x 4 meters), the platform for preliminary large-scale assembly of the hydraulic turbine parts (12 x 18 x 10 meters), a temporary building for the manufacture of precast reinforced concrete members (12 x 6 x 3 meters) and reinforcing steel skeletons (11 x 10 x 5 meters). Almost all of these buildings were constructed out of standard heated panels.

In close proximity to the block being constructed was a pumping station which pumped water into the sea from the drainage system, which surrounded the platform at the bottom of the foundation pit.

The foundation on which the block was being constructed was a rectangular platform (at the bottom of the foundation pit) 50 x 100 meters in size with a packed draining layer of sand (1.2 meters), above which was a layer of 10-25 millimeter crushed stone fragments (0.5 meters) with a layer of 5-10 millimeter crushed stone on top (5 centimeter layer). Within the perimeter of the block itself (18.3 x 36 meters), levelled impermeable plates were installed over the top layer of crushed stone in order to prevent the grout from running out during concreting of the plate.

This compact disposition of all of the ancillary buildings and equipment made it possible to organize the work efficiently with minimal outlays for internal transport. After the floating block was removed from the foundation pit the area was handed over, as contemplated in the project, for future use by the fishing industry.

/178/

The workers were brought to the site from their homes in Murmansk and surrounding villages by municipal buses, which passed by Cape Prityka, and by scheduled launch travelling from Murmansk to Drovyanoe. -At the beginning and end of the working day there was a bus provided by the Northern Administration for Hydraulic Engineering Construction. Lunches were delivered to the site. Water was delivered by tank trucks and was kept in a 100 m^3 reinforced concrete reservoir.

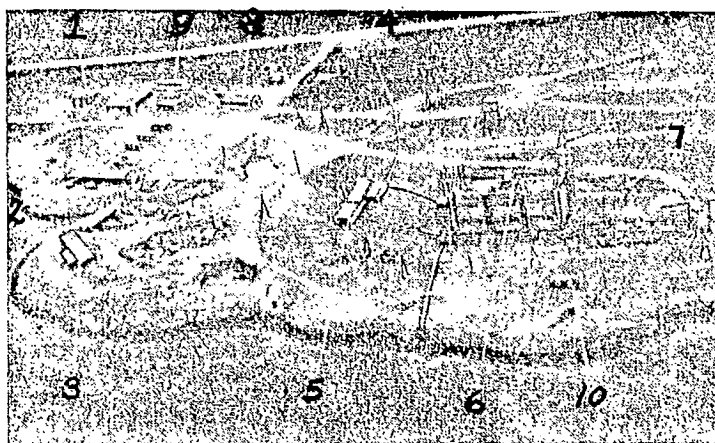


Рис. 8-2. Общий вид площадки стройдока для сооружения наплавного блока здания Кислогубской ГЭС. 31/VIII 1967 г. Фото И. Н. Усачева.

1 — бетонное хозяйство (бетонный завод на две бетономешалки по 250 л, склады цемента на 40 т заполнителей площадью 250 м²); 2 — арматурная мастерская со складом арматуры; 3 — опалубочная мастерская 5×4×30 м со складом пиломатериалов и опалубочных щитов; 4 — железобетонный резервуар с пресной водой и водопровод в котлован; 5 — хозяйство для приготовления и хранения эпоксидной теплогидроизоляции с мастерской и складами готовой продукции (5×4×15 м) и компонентов (5×4×12 м); 6 — блок ПЭС в котловане дока в процессе строительства (по периметру блока строительные леса, с обеих сторон блока — строительные башенные краны); 7 — монтажная площадка дока (в зоне действия кранов); 8 — контора строительства (16×10×3 м); 9 — бетонная лаборатория; 10 — передвижной вагончик — контора нач. участка.

Figure 8-2. General view of the site of the construction dock for building the floating block of the Kislaya Guba TPP structure. 31 August 1967. Photographed by I.N. Usachev.

1 - concrete works (concrete plant consisting of two 250 liter concrete mixers, cement stores for 40 tons of fillers 250 m² in area); 2 - reinforcement shop with reinforcement storehouse; 3 - casing shop, 5 x 4 x 30 meters with supply of lumber and casing grades; 4 - reinforced concrete reservoir with fresh water and a pipeline to the foundation pit; 5 - works for the manufacture and storage of epoxy resin hydrothermal insulation with a workshop and a storehouse for the finished product (5 x 4 x 15 meters) and the raw materials (5 x 4 x 12 meters); 6 - TPP block in the foundation pit dock in the process of construction (there is scaffolding about the perimeter of the block and tower cranes on both sides of the block); 7 - the dock assembly platform (within the zone of operation of the cranes); 8 - the construction office (16 x 10 x 3 meters); 9 - the concrete laboratory; 10 - a moveable trailer, the office of the Section Chief.

The disposition of the temporary structures at the Kislaya Guba site was determined by the general plan of the water engineering system and the sequence of erection of the structure. First a landing barge was used to deliver a compressor, bulldozer, motor vehicle, mobile crane, tents and an office trailer, with the aid of which, in 1964, temporary living quarters and a dining room were set up and work was begun on building approaches to the embankment. A permanent road was then built to the lake which provided the drinking water, where work was going on to install a water intake, a pumping station, to lay a water line and to build pressure tanks. Initially, all light loads arriving at Kislaya Guba were unloaded at the existing piled wharf in the Kislaya Guba basin, which restricted the draft of ships passing through the neck, and also restricted the unloading time of the ships to high water. In order to speed up the operations of working the quarry, building the roads, planning, and the construction of a house, in June 1965, with the aid of a 100-ton floating crane specially assigned to Kislaya Guba (Fig. 8-3), a large excavator, a bulldozer, two motor vehicles from the Minsk Automobile plant, a compressor and a portable diesel electric generating station, a bathhouse; and another staff headquarters trailer was installed for the Section Chief. Living quarters (with the exception of two winterized tents) were not built because the divers and blasters lived on the floating crane and the rest of the personnel (the builders of the electric power transmission line, the drivers of the motor vehicles, bulldozers and excavator operators) were billeted in available apartments at the fish-plant village (Port-Vladimir). A 150 horsepower passenger and tug launch, the "Gidrostroitel", was used for communication with Port-Vladimir, the settlement of Ura-Guba, and for transporting the main loads by water (including towage of barges from Ura-Guba and Chan-Ruchei). The launch was later equipped with radar, which ensured its operation in times of fog and during the polar night.

/179/

The delivery of freight and the transport of construction personnel to the Ura-Guba wharf were handled by the Northern Administration for Hydraulic Engineering Construction.

An important role in ensuring the successful progress of construction, the rapid and flexible manoeuvring of mechanisms, transport, and teams of specialists, which was made especially difficult by the fact that construction was carried out at two different isolated sites separated by the sea and, at times, by roads closed by blizzards, was played by the efficient and reliably operating system of communications. The construction management personnel were in contact, through regional lines, with all subsections and management of the Northern Administration for Hydraulic Engineering Construction and, through Murmansk and Kola, with subcontractors and municipal and regional organizations. Taking into account the uniqueness and complexity of construction, the regional communications administration provided top-priority communications with the Ministry of Electric Power, USSR and with Moscow, Leningrad and Baku, cities in which institutes taking part in the experiment were located. The need to implement a series of new solutions at the Kislaya Guba TPP construction site required that the author's supervision not only ensure scrupulous execution of the designers' ideas but also ongoing adjustment of the project, which is unavoidable during construction of a unique structure by a new method. Thus, when it was discovered during concreting that, as a consequence of spreading of the casing, the weight of the structure would be greater than the design weight, in order to maintain the original draft it was decided, without any delay, to replace the reinforced concrete decking at the 42.9 meter mark and partially at the 46 meter mark with metal decking. With similar dispatch and success project changes were carried out with respect to a decrease in the guaranteed water depth during removal of the block from the /180/

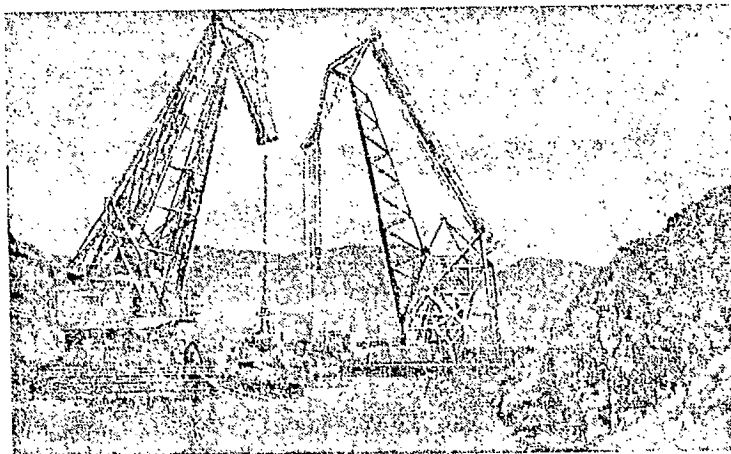


Рис. 8-3. Выгрузка тяжелого оборудования в губе Кислой. 22/VI 1965 г. Фото Л. Б. Бернштейна.

Figure 8-3. Unloading of heavy equipment at Kislaya Guba. 20th June 1965. Photographed by L.B. Bernshtein.

construction site, replacement of anti-fouling coatings on the side walls with fiber glass fabric, replacement of the hydrophobic soil above the variable water level zone with coarse gravel, etc.

/181/

Efficient organization of the author's supervision, its mobility thanks to the presence of the communications system and a special motor vehicle, made it possible for a small working designers' group¹ to provide not only the usual quality control of operations and execution of the project but also, in view of the fact that the Chief Project Engineer himself directed construction, to carry out technical supervision of the work in the most complex and important sections (for example, the underwater excavation, installation of the underwater foundation, lowering the water level during excavation of the construction dock, application of the anti-fouling coatings, ballasting, removal and launching of the floating dock, and much else). The concreting operations were especially interesting and educational

¹Chief Engineer - I.N. Usachev, Supervisor of the Kislaya Guba site - Engineer S.L. Gel'fer, Chief Engineer of the Work Organization Plan - N.N. Nekhoroshev, Supervisors of the Concrete Works - Engineers A.M. Pirogov and A.Ya. Vinogradov, Senior Engineer L.I. Nekhorosheva, Technician T.Ya. Nekrasova and Mechanic-Operator A.I. Tochilin, Supervisor of the Under Water Technical Operations - Section Chief, Engineer V.G. Gavrilov.

in this regard. To ensure the quality of these operations and the personal responsibility of each participant use was made of concreting control diagrams (Fig. 8-4).

а- Бетонную смесь по зонам стены уплотняют вибраторами.

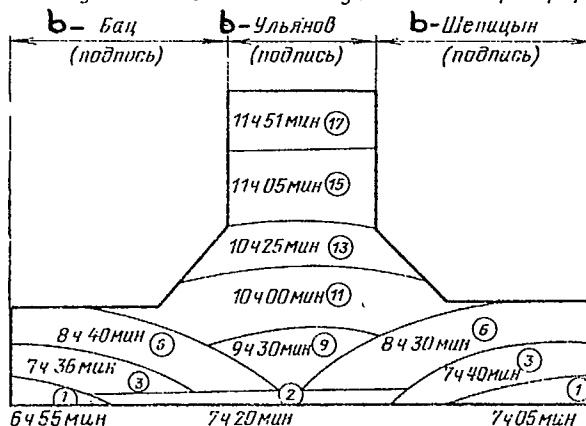


Рис. 8-4. Контрольная карта бетонирования стенки блока Б 1-3, первого яруса 3/IX 1965 г. в смену от 7 ч утра до 13 ч 30 мин бригадой М. Марквартова при кураторе ГРП А. Я. Виноградове и прорабе Е. И. Макарове.

Подача бетонной смеси осуществляется вручную через верх блока. Бетонная смесь уплотняется глубинными вибраторами. Цифры в кружках означают номера рейсов автосамосвалов с бетонной смесью (регистрируются на бетонном заводе и в котловане). Время — начало подачи бетонной смеси в блок в данный участок.

Figure 8-4. Concreting control diagram of the block wall B1-3, first level, 3 September 1965, during the 7:00 am to 1:30 pm shift, by M. Markvartov's team under the supervision of A.Ya. Vinogradov and Foreman E.I. Makarova.

a - vibrator operators consolidate the concrete mixture by zones; b - workers' signatures.

The concrete mixture is delivered manually through the top of the block. The concrete mixture is consolidated using internal vibrators. The circled numbers indicate the sequence of trips by the dump cars with the concrete mixture (recorded at the concrete works and at the foundation area). The times shown indicate the start of delivery of the concrete mixture to the block in the given section.

Chapter 9

CONCRETING OPERATIONS

9-1. Choice of the Concreting Method

In carrying out the concreting operations the following requirements were present: to ensure high placement quality of the stiff concrete mixture (2 centimeter slump workability) into the openwork TPP structure, whose main member is a 15-centimeter reinforced concrete wall reinforced with two 10 x 10 centimeter grids (5 x 10 centimeters in the variable water level zone) made of 25G2C type deformed steel, 10 millimeters in diameter, with a 2 centimeter thick protective layer. In order to solve this problem it was necessary to examine various methods of concreting thin-walled members.

The precast and monolithic methods of concreting were examined first. With mechanized concreting of the members in the horizontal position, although the precast variant decreased the time needed to erect the TPP, it required equipment with greater lifting capacity per block of the TPP structure and, although using a small total amount of concrete, would have required 69 different types of members and only a few of each type. For this reason the monolithic method was chosen. In order to gain experience in the manufacture, assembly and monolithification of precast members for thin-walled structures, however, which will be required during the construction of powerful tidal power plants, a small number of precast members were manufactured. /182/

The monolithic variant could have been executed in several ways: by batch delivery of the concrete mixture to the structure, with the aid of concrete pumps, by the discrete method, and by placement of the concrete onto the surface of the structure under pressure.

The delivery of a stiff concrete mixture with the aid of pumps did not appear to be possible because such a mixture cannot conveniently be delivered by pipes.

The discrete method of concreting, with the delivery of an activated mixture into a block casing filled with crushed stone, despite the possibility of achieving high density and impermeability, turned out to be inexpedient under the given conditions due to the difficulty of high-quality injection of the mixture into the thin-walled TPP members and the need to use very small fraction crushed stone.

The spray concreting method (gunite or shotcrete) also turned out to be inapplicable because, during mixing of the dry mixture with water, a non-uniform mixture formed which segregated into layers when it was applied to the surface.

Thus, the most suitable method for the Kislaya Guba TPP was found to be the method of batch delivery of the concrete mixture.

The experience of concreting thin-walled structures was accumulated during the construction of reinforced concrete ships, in river and harbor construction, during the construction of high thin-walled reinforced concrete structures and in the monolithification of seams between precast members in hydraulic engineering. In the above structures, however, concrete mixtures with slumps of 6-19 centimeters were employed, with metal forms and consolidation of the concrete mixture through the form with the aid of external vibrators. With this method, however, the quality of the concreted members was not always good. Thus, the frameworks of reinforced concrete docks showed defects over 5-10% of their total areas, and in the walls of certain high structures significant nonuniformity of the composition of seams and in cross section has been observed.

Experimental concreting of fragments was carried out in order to choose the techniques of placing still concrete into thin-walled TPP structures.

In concreting the TPP fragments, various types of forms were used: /183/ solid with horizontal and vertical openings for concreting, and out of assembled panels, and out of various types of materials: wood, metal and wood with the working surface faced with water-retaining cardboard (soft and stiff) in order to adsorb excess water from the peripheral sections of the members during their consolidation.

The main criterion in the selection of forms was the need to ensure a reliable concrete surface layer, uniformity of the concrete throughout the entire cross section of the wall and maintenance of a 2-centimeter thick protective layer.

When a form with a metal surface was used an excess quantity (5%) of water was found in the peripheral wall layer. When a form with a metal and wood surface was used cavities due to air bubbles were observed on the concrete. It was only possible to eliminate these bubbles by facing the forms with water-retaining soft cardboard (Table 9-1). Use of pre-assembled forms made it impossible to maintain the integrity of the protective layer.

Table 9-1

Description of damage to the concrete surface depending on the form material (concrete with sulfite-alcohol distillery waste and air-entraining neutralized resin)

Form material	Size of cavities, cm, <u>diameter</u> depth	Number of cavities (nominal per unit of area 20 x 20 cm)
Metal	0.15-4.0	7-15*
	0.7 - 2.0	
Wood	0.3-1.2	5-10
	0.4-1.2	
Cardboard (soft)	None **	

* When gaseous silicone liquid (GSL) is used as an additive the number of cavities doubles.

** When stiff cardboard is used the cavities are not more than 1-2 millimeters deep and up to 5-10 centimeters in diameter.

The forms also had to maintain the design cross section of all of the structural elements in order to ensure the planned draft of the block during towage. For this reason, in order to disallow spreading, the form panels were connected with bolts because the use of clamps through the studs, even when they are tightened to 0.95 of the design thickness of the member, does not guarantee elimination of spread. The protective layer was made possible by installation of concrete liners between the forms and the reinforcing mesh (4 per one m^2); the reinforcing mesh sections were rigidly attached to each other because the flexible reinforcement grips used in reinforced concrete shipbuilding resulted in unequal spread of the walls between the bolt fasteners.

/184/

Experimental concreting of the fragments showed that wooden form sections with horizontal openings and with soft cardboard facings on the working surface (Fig. 9-1) were the most suitable for placing the concrete in the blocks. They ensured uniformity of the mixture throughout the wall, high quality of the external surface and observance of the design cross section of the structure, as a result of which the weight of the block when it was floated was equal to the design weight.

In order to ensure uniformity of the concrete mixture, stability of its consistency and the required amount of air, proper batching must be observed during preparation of the concrete mixture. The engineering requirements for the concrete at the Kislaya Guba TPP required precise batching of the cement, water and additives using 1% of crushed stone and 2% sand. Because there is no batching equipment of the required precision four small concrete works, a manual weight batching method was used for the cement, sand and crushed stone and a volume method for the water and additives. A mixing time of four minutes for the concrete mixture was determined

/185/

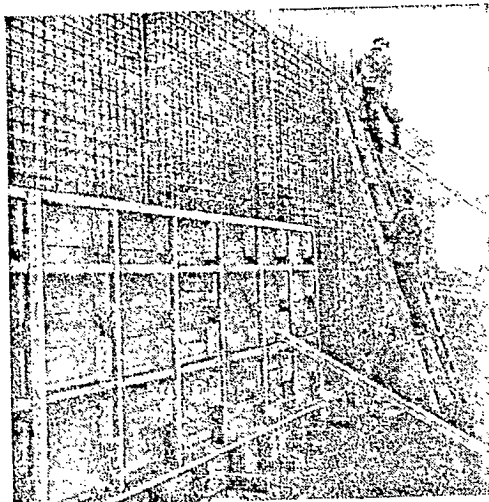


Рис. 9-1. Опалубочные щиты и армосетки
в блоках бетонирования Кислягубской
ПЭС. 29/IX 1965 г. Фото А. С. Фирфарова.

Figure 9-1. Form panels and reinforcing mesh in the concrete blocks of the Kislaya Guba TPP. 29 September 1965. Photographed by A.S. Firfarov.

by controlling mainly for air content as well as for uniformity and slump (the mixing times were later established for a range of temperatures between 2 and 20°C).

The main requirement in transporting the concrete mixture consisted in preserving uniformity of the mixture, not allowing the water to separate and layering to occur or to lose workability of the mixture during loading, transport and placement into the block. In winter the mixture could not be allowed to cool below a temperature of +5°C and, finally, in order to maintain high frost resistance, loss could not be permitted of the air entrained within the mixture below the level permitted by the engineering requirements.

Changes in the uniformity of the mixture (in terms of content by weight of the large filler) was controlled during transport by wet sieving of samples of the mixture taken from the bucket and the block. Data-spread from these tests fluctuated within the limits of 1-5%. Thus, it may be assumed that layering of the concrete mixture did not occur during transport. Sampling of the concrete after consolidation, from various points vertically along the block, showed deviation of water content of not more than 2.5% and it follows from this that separation of water did not occur during transport either.

The effect of transport conditions on the quantity of air in the concrete mixture was determined. Most of the air is drawn into the mixture during addition of the air-entraining additive and during mixing. Transport of the mixture and depth of drop during placement had little effect on changes in the amount of air.

When the concrete mixture was placed into the block it was necessary to preserve uniformity of the mixture and to disallow layering of it. The depth of free drop of concrete is limited by standards to 2 meters, while delivery through elephant trunks with an inner diameter of not less than three times the size of the largest diameter of the large filler is restricted to 10 meters. For greater heights delivery of the mixture is recommended only through vibrating trunks. These rules were observed during construction of high reinforced concrete reservoirs in which a concrete mixture with a slump of more than 6 centimeters in the wall was delivered by trunks to a depth of 4-7 meters. During sealing of joints, drop is known through a telescopic trunk in a 60 centimeter opening to a depth of 11 meters; during concreting of the pile forms, concrete was delivered through a duct to a depth of 18 meters [22]. Lack of segregation of the concrete mixture

during its free drop within the wall to a depth of 3 and even 7 meters during concreting of the TPP block may be explained, on the one hand, by the reinforcing mesh forming a unique elephant trunk, and on the other hand, by the presence of air bubbles in the concrete mixture which homogenized it and prevented segregation.

The choice of the consolidation method, the type of vibrating mechanism for consolidation, and determination of the optimum consolidation regime were made experimentally. During construction of the Kherson reinforced concrete wharf [24] it was found to be impossible to consolidate a stiff mixture with the aid of external clamp and bayonette vibrators. When internal vibrators were used for consolidation the best results were obtained with vibrator I-116A (subsequently, when working in blocks 5-7 meters deep, the length of the vibrator's flexible shaft was doubled, which decreased the output of the vibrator by 30-40% but did not damage the quality of consolidation) and the "Vibro" vibrator (Swedish model) with a 7-meter long shaft. The pneumatic internal vibrators S-697 and S-698 were not capable of consolidating the stiff mixture and, in addition, they cannot operate reliably at temperatures below freezing.

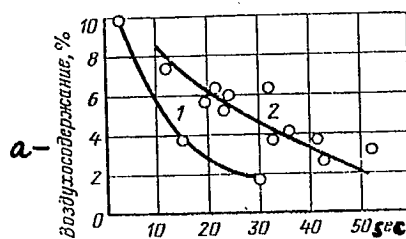


Рис. 9-2. Содержание воздуха в бетонной смеси в зависимости от длительности ее вибрации.

1 — по опытным данным ИНИИС;
2 — в блоках бетонирования (вibrаторы И-116А и «Vibro»).

Figure 9-2. Air content in the concrete mixture as a function of the length of time it was vibrated.

1 - according to experimental data of the All-Union Scientific Research Institute of Transportation Construction; 2 - within the concreting blocks (I - 116A and "Vibro" vibrators); a - air content, %.

The optimum consolidation regime (radius of action of the vibrator and duration of vibration) was determined experimentally, taking into account the need to maintain the minimum necessary amount of air in the mixture needed to ensure high frost resistance (Fig. 9-2).

The question of using the reinforcement to vibrate the underlying layers (which is unavoidable considering the distance between the reinforcing mesh is 6 centimeters), required especially close examination. /187/

Experimental data on the attenuation of vertical oscillations in concrete [54] show that oscillations at a frequency of 3,440 cycles per minute damp out at a depth of 35-40 centimeters. At a frequency of 1,200 cycles per minute it may be assumed, by comparing coefficients of attenuation in a concrete mixture for the given frequencies and amplitudes, that the oscillations are damped out at a depth of 80-90 centimeters. Inasmuch as the rate of placement of the concrete in the wall was 1.5 meters per hour (when the concrete was placed in a long wall, up to 30 meters in length, by the inclined slopes method), and the setting time for the given engineering conditions was three hours, such oscillations were not dangerous.

The loss of workability of the mixture during long periods prior to its use (taking into account delays during transportation at high or exceedingly low temperatures) may apparently be explained by the loss of a certain amount of water during transport. This results in the so called "critical" amount of water in a stiff mixture (according to the research of the All-Union Scientific Research Institute of Transportation Construction) equal to approximately 160 liters per m^3 for standard vibration, below which workability deteriorates sharply. Attempts to work such a mixture by means of lengthy vibration only result in its layering.

The high uniformity of the mixture in terms of the large filler (from samples taken in various parts of the block after consolidation) and in terms of strength (determined during construction) indicate the high quality of consolidation of the concrete mixture by the internal electric vibrators I-116AM and "Vibro" when their duration of operation at individual points was within the limits of 20-40 seconds with an effective radius of 20-30 centimeters for a placed layer thickness of 10-35 centimeters.

Experimental concreting revealed that, following the developed techniques, a stiff concrete mixture could be placed in thin-walled densely reinforced blocks of the Kislaya Guba TPP structure while maintaining high concrete quality.

9-2. Erection of the TPP Structure

a) Concreting

Concreting of the floating block (the "hull" in shipbuilding terminology) was done in the construction dock at Cape Prityka, where the necessary support facilities were closely situated near the channel and the "Prichal'noe" construction base of the Northern Administration for Hydraulic Engineering Construction (see section 8-3).

/188/

The TPP block was built with the aid of two tower cranes with a lifting capacity of 5 ton-force, located on either side of the block. Stock metal scaffolds with adjustable spans were installed between the block and the crane tracks for performing the concreting and waterproofing operations.

The 36 x 18.3 x 15.5 meter TPP block, with a concrete volume of 1,800 m³, was divided into three vertical levels as required by the use of the vibration consolidation equipment. The levels were divided into 72 concreting blocks (with an average volume of 25 m³) with a single type of

structural member in each block (bottom, external walls, internal bulkheads, suction pipes, deckings and piers), in order to ensure convenient placement of the concrete and the time required by the engineering conditions for covering the concreting layers.

The form work for the monolithic variant was found to be very time-consuming and expensive (labor outlay per 1 m³ of concrete - 10 man-shifts or 39% of the total expenditure for concreting operations). The cost of the forms comprised 40% of the actual cost of the concrete for the job, due to the high modulus of the forming surface for the thin-walled TPP structure (14.0 m² per m³) and the extensive consumption of lumber (0.08 m³ per m²), in order to ensure high rigidity of construction, and the complexity of linkage. In addition, due to the climatic conditions, a large part of the forms remained on the block for thermal protection, which resulted in a low rate of reuse.

Understandably, this high form-work cost determined, to a significant extent, the high cost of the concrete for the Kislaya Guba TPP. It is exclusively explained by the specific nature of the experimental structure, for whose small dimensions it was impossible to apply modular construction with mechanized concreting of the members in horizontal molds.

The reinforcement work consisted in installing double 10 x 10 centimeter mesh into the block. Manual installation of the reinforcing mesh in the block for the first and second levels was found to be inconvenient, for which reason the meshes for the upper level were assembled on the slopes of the foundation pit, after which they were installed in the block and welded. The installation periods for the meshes and the forms covering them had to be regulated because, if the block was assembled too long before concreting, the reinforcement rusted and it was necessary to remove the forms in order to clean it.

The labor required to prepare and assemble the reinforcing mesh was 4 man-shifts per 1 m^3 of placed concrete. The reinforcing operations took up 18% of the total working time on the block and their cost comprised 25% of the cost of concreting operations with an average expenditure of 150 kilograms of reinforcement per 1 m^3 of concrete [44].

The concrete mixture was prepared in two 250 liter concrete mixers with manual batching of the aggregates. The actual productivity of the manufacture of the concrete mixture was $1\text{--}2.5 \text{ m}^3$ per hour, which ensured constant placement of the concrete in accordance with the times required for covering successive layers. The concrete mixture from the concrete works was transported by dump trucks over a distance of 400 meters. The mixture was delivered to the concreting location by means of a "Kama" bucket with a capacity of 0.7 m^3 , equipped with an easily operated segmented gate with a vibrator on the body. In the block the mixture was dropped to a 3-meter depth through a 6-centimeter space between the reinforcing meshes through openings in the forms and the top of the block.

In order to distribute the concrete mixture within the block of long (longer than 10 meters) thin-walled members, which were more than 3 meters deep, it was placed in the form of a "ladder", which was found to be more efficient in comparison with frontal distribution of the mixture. With this method less labor and vibration consolidation mechanisms were required, there was a significant decrease in the length of time required to cover the layers and there was an increase in the efficiency of concrete placement control. This method was used to concrete almost all of the TPP block walls.

During concreting, careful control was kept over maintaining the workability and uniformity of the mixture and, for blocks in the variable water level zone, of the quantity of air required to ensure high frost resistance.

The rate of placement of the concrete was 1.5-2.5 m³ per hour, the labor costs during placement of the concrete were 0.7-1.6 man-shifts per 1 m³; the concrete placement cost comprised 7% of the cost of concreting operations.

Concreting of the TPP block was begun on 31 July 1965 and completed in August 1967. The bottom plate (Fig. 9-3) was concreted on 3 August 1965. /191/ The walls of the first level (Fig. 9-4) were erected in the fall of 1965 at temperatures below freezing and were kept under canvas tents. In the winter of 1965-1966, in a heated enclosure, the central part of the TPP block was constructed, which made it possible to begin assembly of the hydraulic turbine unit and the lower half of the suction pipes (Fig. 9-5). In the summer of 1966 the upper half of the suction pipes and the blocks of the second level were concreted (Figs. 9-6 and 9-7). In the winter of 1966-1967 the walls of the service compartments were completed under local heated enclosures and in the summer of 1967 the third level was completed. Thus, concreting of the TPP block took two years instead of three summer months called for in the project. This delay was caused by various organizational breakdowns during the initial period of construction: lack of the required number of workers at the site; work conducted mainly during one shift (although summer conditions in the arctic made it possible to organize work around the clock), non-availability of cement (for three months in 1967); non-fulfilment of important design recommendations (thus, as a consequence of the rejection of a full heated enclosure during the winter of 1966-1967, it was necessary to spend time on cleaning the reinforcement which was installed during the winter and on repeatedly moving the local heated enclosures).

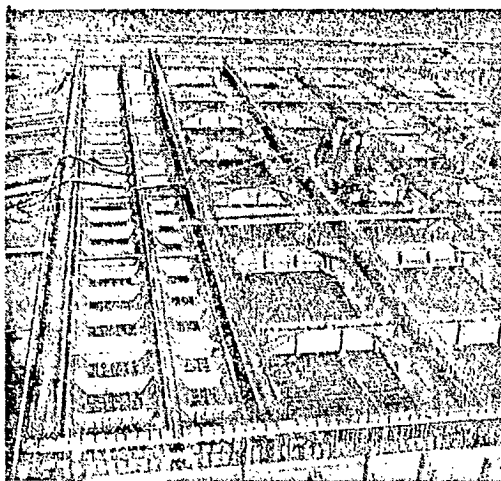


Рис. 9-3. Блок динищевой плиты перед бетонированием. Июль 1965 г. Фото А. С. Фирфарова.

Figure 9-3. Bottom plate block before concreting. July 1965. Photographed by A.S. Fifarov.

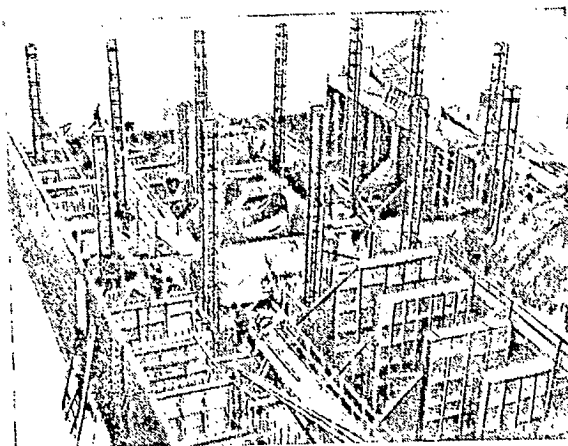


Рис. 9-4. Бетонирование первого яруса. 18/X 1965 г.
Фото А. С. Фирфарова.

Figure 9-4. Concreting of the first level. 18 October 1965. Photographed by A.S. Firfarov.

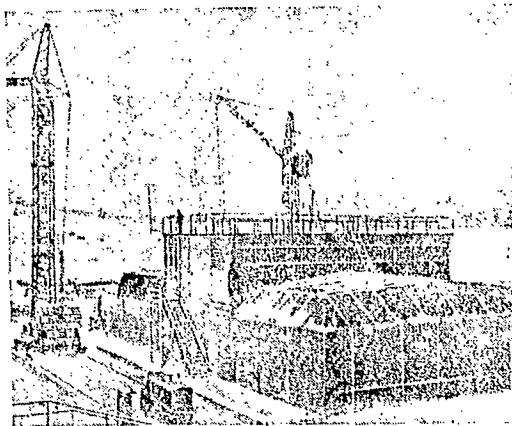


Рис. 9-5. Бетонирование агрегатной части и
отсасывающих труб ПЭС в тепляках. 3/XII
1965 г. Фото А. С. Фирфарова.

Figure 9-5. Concreting of the turbine unit part and the suction pipes of the TPP under heated enclosures. 3 December 1965. Photographed by A.S. Firfarov.

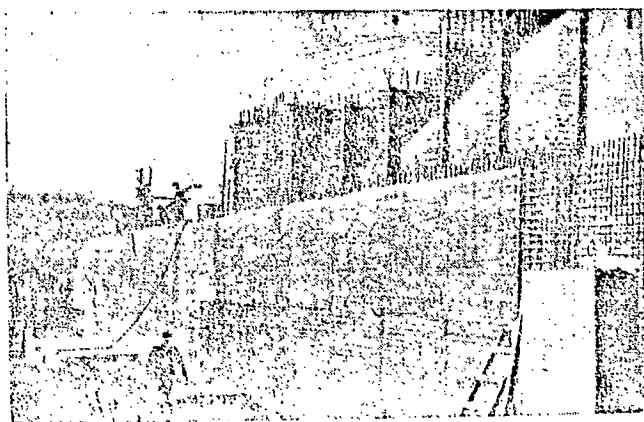


Рис. 9-6. Возведение второго яруса. 18/V 1966 г. Фото
Л.Б. Бернштейна.

Figure 9-6. Erection of the second level. 18 May 1966. Photographed by L.B. Bernshtein.

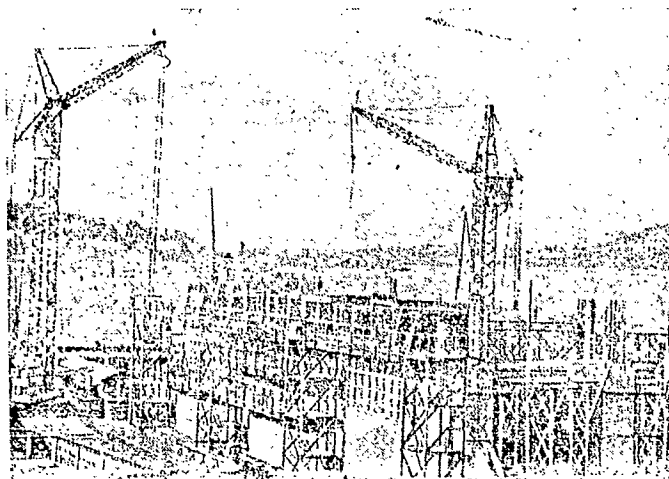


Рис. 9-7. Распалубленный бетон первого яруса и отсасывающих труб с поперечными переборками. 1/VIII 1966 г. Фото Л. Б. Бернштейна

Figure 9-7. The concrete of the first level, the suction pipes and the transverse bulkheads with the forms removed. 1 August 1966. Photographed by L.B. Bernshtein.

b) Precast Concrete

In view of the fact that industrial TPPs must be built using only precast reinforced concrete, in order to examine possible techniques (including sealing of seams), part of the upper level (pier bulkheads) of the Kislaya Guba TPP was built using precast members with looped reinforcement (Fig. 9-8).

/192/

/193/

Precast reinforced concrete ensures not only the high quality of thin-walled TPP members, concreted horizontally, but also (for structures of this type) greatly lowers the cost of concreting operations (decreases the cost of formwork ninefold, which comprises 40% of the total cost of concreting operations for the monolithic variant).

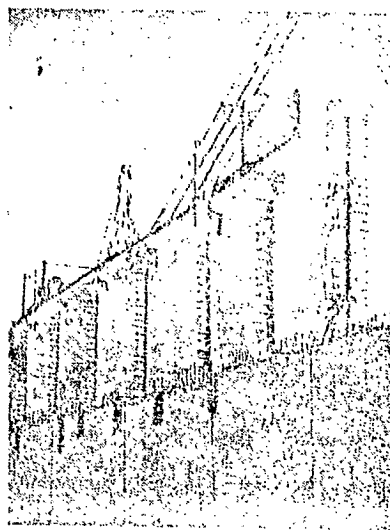


Рис. 9-8. Сборные железобетонные
элементы с петлевым усилением.
Апрель 1967 г. Фото И. И. Усачев.

Figure 9-8. Precast reinforced concrete members with looped reinforcement.
April 1967. Photographed by I.N. Usachev.

The main difficulty in the use of precast reinforced concrete consisted in ensuring the high quality of joint seals.

The vertical seam (six meters deep) was sealed through openings in the formwork located 80 centimeters apart; the horizontal seam was sealed through the horizontal slot in the formwork for admitting the vibrator heads. In this case the slot was located 3-4 centimeters above the upper end of the precast member. The ends of the precast members were prepared with the aid of a small pneumatic hammer. Concrete of the same composition as that used in the TPP block was used for the precast members and the joint seals. The concrete in the joint seals was consolidated with the aid of improved I-116AM internal electric vibrators. The consolidation regime was similar to the regime used throughout the block.

Tests showed uniformity of the mixture (in terms of large filler content and strength) throughout the extent of the seam seals, and their complete impermeability.

The precast members were cast in wooden molds with the concrete mixture spread in one layer for the entire thickness of the member and consolidated with a I-116A vibrator. In summer the members were concreted in the open, in winter they were manufactured in a building heated to $+10^{\circ}\text{C}$. The members were not heat treated (according to the engineering specifications for the concrete at the Kislaya Guba TPP, heating above 40°C was not permitted) in order to avoid a decrease in frost resistance.

/194/

c) Concreting in Winter

The project called for concreting of the TPP block during the course of a single summer. As a consequence of the above mentioned organizational breakdown, however, concreting operations were also carried out in winter. The impossibility of introducing various anti-freeze additives, which would decrease the frost resistance of the concrete, and the use of peripheral electric heating due to the danger of drying out the protective layer, as well as the inefficiency of the "thermos" method for thin-walled members, predetermined the use of tents and heated enclosures, which greatly complicated the work.

In autumn and spring, at air temperatures down to -7°C , concreting was performed in the open air with subsequent curing of the block under a canvas tent at a temperature of $5-10^{\circ}\text{C}$. At lower air temperatures concreting and curing were carried out in heated enclosures of insulating material over a lathwork (Fig. 9-5). Concreting operations were suspended at temperatures below -15°C . As a result of heating the water to $+50^{\circ}\text{C}$ and the sand to 10°C the concrete mixture, at an air temperature of -15°C , had

a temperature of 15-20°C in the concrete mixer and a temperature of 5-9°C in the block after completion of concreting. During curing of the concrete, at an air temperature of -20 to -30°C, the temperature of the concrete was 10-12°C (+22°C maximum), and the minimum temperature in the heated enclosure was 3-6°C.

The concrete placed in winter (50% of the total volume), was of a high quality thanks to the strict control of the author's supervision with respect the adherence to the temperature regime of the concrete.

d) Concreting Defects and Their Elimination. Waterproofing

The above mentioned (section 8-2) organizational breakdowns during the initial period of construction resulted in a disruption of the concreting process and a decrease in the quality of operations. In some concreting blocks of the suction pipes and the inner walls, cavities were discovered after removal of the formwork as a result of the fact that the specified thicknesses of the placed layers of the concrete mixture were significantly exceeded and the fact that it was insufficiently consolidated in individual sections. These cavities, in violation of the project recommendations for repair, were patched with a cement solution and, during the period of operation after flooding of the suction pipes and the ballast spacings, gave rise to three leaks which had to be connected to the drainage system. /195/

Six blind cracks up to 150 microns wide and 2.5 meters long appeared on the outer surfaces of the block. This occurred as a result of the violation of the thermal regime of curing the structure in winter (a heated enclosure was partially open). One of the cracks was found to be due to local overheating of the concrete by the electric heaters, and another as the result of the rupture of a small structural member when water froze in

a closed cavity which was entrapped during construction. The most significant defects, several cavities extending completely through the concrete during concreting in July 1967 of the west end wall, were caused by insufficient mixing of the concrete (but not as the result of stiffness of the concrete, as it was established from laboratory data that concrete placed earlier was of the identical stiffness and was of high quality).

The defects were eliminated in accordance with specially developed and field tested recommendations of the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute: the cavities extending completely through the members were patched with the basic concrete mixture and the seams were treated with an impermeable epoxy resin binding compound, while the small defects were patched with an epoxy resin glue with a sand filler.

Attachment to the concrete of the embedded members for anchoring the pipes of the cementation system and the side cages for the pontoons, in order to avoid damaging the walls, was carried out in accordance with the recommendations of the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute which specified that an epoxy resin compound be used to glue to the concrete surface of the block concrete cubes with projections for mounting. During the process of equipping the TPP block with pontoons and during the process of constructing the connecting dams, almost all of the glued cubes were torn off, but examination of the break revealed that it had occurred in the concrete and not in the glue seam.

Waterproofing of the construction seam between the underwater zone of the TPP block and the zone of variable water level, in order to exclude capillary suction of water into the variable water level zone, was achieved by means of two layers of epoxy resing glue: the first layer was brushed on after removal from the seam of a cement film and prior to work on the overlying block, the second layer was applied prior to placement of the concrete.

/196/

e) Cost of Concrete

The unit cost of the concrete for the Kislaya Guba TPP, as worked out prior to construction, was determined to be 200 rubles. The actual costs were 122 rubles per m^3 , which is in agreement with the cost of reinforced concrete for hydroelectric power plant construction carried out by the Murmansk State Marine Construction Office. Analysis of the execution of concreting operations at the Kislaya Guba TPP construction site reveals that this cost can be lowered during the construction, on an industrial scale, of thin-walled structures composed of precast members by using standard formwork with a high reuse capacity and by mechanizing the concrete works.

Thus, the experience of concreting the Kislaya Guba TPP block proved the possibility of erecting thin-walled floating structures, powerful tidal power plants and hydroelectric power structures, under the conditions of northern tidal seas, out of very highly frost resistant, low-plasticity reinforced concrete.

The techniques of preparing and placing the concrete which were developed and applied at the TPP construction site showed that high quality concreting of a thin-walled structure by the monolithic method could be ensured using even low-plasticity concrete. However, this method, which was used due to the small size of operations at the experimental TPP, must give way, on an industrial scale, to the precast method under which the horizontal concreting of the elements and their large "run" simplifies and facilitates the process and ensures high concrete quality, which is substantiated by the experimental construction of individual members of the thin-walled TPP structure out of precast reinforced concrete. It is especially important to note that full-strength and waterproof seam seals were obtained with this method.

Epoxy resin compounds for waterproofing structural seams, which were used in the construction of the TPP, ensure the impermeability of the joint but must be time tested.

Chapter 10

UNDERWATER ENGINEERING OPERATIONS

10-1. Formation of the Underwater Foundation Pit

a) Excavation

The underwater excavation began in May 1964 with the breakup of oversized boulders. the stone embankment in the throat of Kislaya Guba had /197/ not yet been constructed at that time, for which reason the work was done only at high water when rate of flow was insignificant. Actual dredging of the bottom began only a year later, after the floating crane "Valmet" and a 100 ton-force dump scow were towed into Kislaya Guba. The performance of the floating crane was high only in May (approximately $3,400 \text{ m}^3$), June and July. In August the performance was lower than expected ($1,000 \text{ m}^3$) even though work was being done on lighter ground. The main reasons for the low performance were the inefficiency of the blasting operations in breaking up oversized boulders and the presence of consolidated layers of ground. The planned solution (loosening of rocky ground underwater by the blast hole method) could not be implemented due to the extreme difficulty of drilling. The detonation of 20-40 kilogram explosive charges was not effective. Furthermore, small explosive charges, which loosened the ground insignificantly, apparently only served to consolidate underlying ground. Further decrease in excavation efficiency necessitated the search for a more effective method of loosening the ground. Study of this problem resulted in the adoption of a proposal to bore holes 200 millimeters in diameter for the blasting operations.

In view of the lack of the necessary drilling technology at the construction site, this work was entrusted to the Third Expedition of the Leningrad Affiliate of the All-Union Planning, Surveying and Scientific Research Institute, which installed equipment for drilling holes 127

millimeters in diameter (instead of 200 millimeters) on pontoons provided for it. Despite the measures taken, drilling was carried out only to a shallow depth; casing was installed in only one of seven drill holes. Drilling resulted only in the discovery of an increase in the percent content of boulders in the ground, which made it necessary to re-examine and increase the unit cost for the excavation.

As a result of the failure of further underwater excavation operations, the Northern Administration for Hydraulic Engineering Construction /198/ proposed to erect the TPP between dams. The Chief TPP Project and Construction Engineer saw this as a complete rejection of building the TPP, since the whole point of the experiment was to prove the possibility of erecting a TPP without dams by the floating method. This was exactly how the problem was presented in October to A.A. Belyakov, Deputy Chairman of the Scientific and Technical Soviet of the Ministry of Electric Power USSR, who had arrived at the site. When this problem was examined jointly with the subcontracting and design organizations the proposal to construct dams was rejected and, taking into account the available technology, it was decided to loosen the ground by means of concentrated pressure explosive charges.

The decision to loosen the ground of the construction pit by means of concentrated pressure charges presented a series of difficulties which included, first of all, ensuring the stability and safety of the structures in the construction area at Kislaya Guba (chiefly the multipurpose building and the wharf). Since the minimum distance from the point of the explosion to the wharf was 40 meters and 70 meters to the building, powerful explosions could result in the destruction of these structures.

In view of the fact that the supervisors of the underwater and blasting operations¹ refused to accept responsibility for possible damage to the structures at the site, general coordination of the operations was handled by the Chief Project and Construction Engineer. The distribution of the explosive charges and authorization to blast were given in writing in each instance by representatives of the author's supervision.

Experimental detonations (Fig. 10-1) were carried out in December 1965. 500 kilogram charges were simultaneously detonated in 500-2,000 kilogram combinations. The purpose of the blasts consisted not only in checking the loosening effectiveness, but also in evaluating the maximum permissible number of simultaneously detonated charges at various distances from the structures. After the completion of the experimental blasts in January 1966 a test excavation of 600 m³ of ground was carried out.



Рис. 10-1. Мощный подводный взрыв для рыхления грунтов в котловане ПЭС. Одновременно взорвано четыре заряда по 500 кг. Январь 1966 г. Фото С. Л. Гельфера.

Figure 10-1. Powerful underwater explosion to loosen the ground in the TPP foundation pit. Four 500 kilogram charges were simultaneously detonated. January 1966. Photographed by S.L. Gel'fer.

¹The blasting operations in the TPP foundation pit were carried out by the Murmansk Administration of the Trust for Drilling and Blasting Operations of the Main Administration for Special Types of Construction and Installation with the participation of diver demolitions men of the Rescue and Salvage, Shipraising and Underwater-Technical Operations Unit.

Testing of the safety of the wharf was carried out with instruments by a special method. The safety of the building was determined visually. The settlement of the wharf which occurred was insignificant (30 millimeters) and undermining of the lower sill was minor. Thus, all apprehension was eliminated.

However, work using this approved method only got underway in April 1966. The regions to be loosened were determined and the sizes of the explosive charges were set. Thus, the explosive charges were set out in the region to be loosened on a 3 x 3 meter or 2 x 2 meter grid, 500 kilogram charges 1,000 - 2,000 kgms, and 13 - 18 tons per series of detonations. According to underwater inspection the depth to which the bottom was loosened was approximately 1.5 meters. /199/

The excavation operations were begun immediately after detonation, as it was found that the underwater soils quickly consolidate and excavation again becomes inefficient. Table 10-1 presents data on the volume of soil removed and the explosive charges used to loosen it during the most intense period of excavation operations. These data determine the average expenditure of explosive charges per 1 m^3 of soil removed from beneath the water.

The average specific expenditure of explosives was $145,000/11,378 = 12.7$ kilograms per m^3 , which differs insignificantly from the (13 kilograms per m^3) assumed on the basis of the experiment.

The non-uniformity of specific expenditures by month is explained by differences in the soils and a certain redistribution of them at times when the loosened soil was not immediately removed and subsequent loosening made it possible to excavate greater volume.

Table 10-1

Underwater excavation of soil from the TPP foundation area

Period (month, year)		Volume of soil excavated (thousand m ³)	Expenditure of explosive charges for loosening the soil (thousand kg)
1966	April-May	1,650	14.7
	June	830	23.0
	July-August	2,700	13.0
	September	703	13.0
	October	495	17.0
	November-December	2,000	15.0
1967	January	850	18.0
	April	2,150	31.3
TOTAL		11,378	145.0

The total loosened volume for the project was 19.28 thousand m³ (the volume excavated was 25.8 thousand m³); the distribution of this volume by categories of loosening is presented in Table 10-2.

/200/

As noted above, loosening by the blast hole method was not carried out and for this reason the actual volume of explosives was slightly above the planned volume.

Table 10-2

Distribution of soils of the underwater foundation area by types of loosening.

Types of loosening	Category of loosening	Volume, m^3 thousand m^3
Loosening of the rocks underwater by the blast hole method (loosening was actually accomplished by small pressure charges)	VIII	5,100
Loosening of rocks above water by the blast hole method	VIII	1,150
Loosening of the foundation area soils underwater by means of concentrated pressure charges	II, IV, VI, VIII	11,290
Loosening the soils of the slopes of the foundation area by means of pressure charges	VI	1,740

This experiment showed that, despite the loosening of the soils by pressure charges, this method cannot be deemed effective for loosening heavy soils for the construction of industrial TPPs. In addition to the large expenditure of explosives per 1 m^3 of soil, this method caused significant caving in of the banks [$3,000 \text{ m}^3$ of the previously noted increase (section 3-2) of the excavation volume].

/201/

The loosening operations using pressure charges and excavation of the soil (Fig. 10-2), which resumed in April 1966, continued without significant changes until October. Subsequently, corrections were made in the placement of the explosive charges in the direction of decreasing the size of the charges and the weight of simultaneously detonated explosives. This decision was necessitated by a decrease in the thickness of the soil layer being worked on 1 m in some places and by significant caving in of the banks.

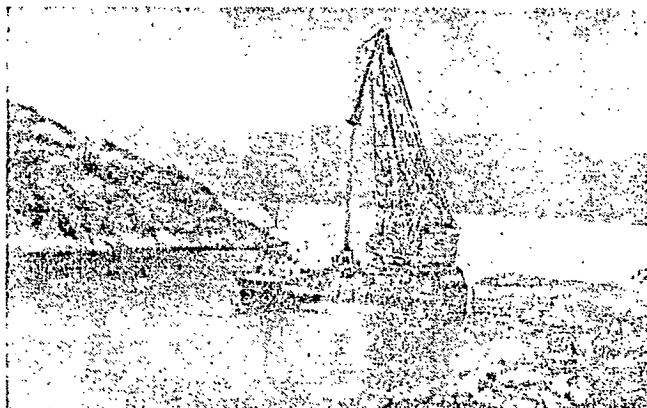


Рис. 10-2. Выемка грунтов котлована ПЭС.

Figure 10-2. Excavation of the soil from the TPP foundation area.

From the middle of January to the middle of March 1967 the floating crane was at Port Vladimir for routine maintenance. Resumed excavation continued with certain technical interruptions until the end of March 1968, after which the crane was towed to Murmansk for repairs. By that time 200 m³ of soil remained to be excavated in the foundation area, work on the sections along the banks for the passage of the crane (after installation of the TPP block at the site) was basically completed, and the slopes and foundation area proper had also been cleaned several times with the aid of a hydraulic excavator with subsequent excavation of the soil using the crane.

Further preparation of the foundation pit was carried out by divers, who loaded and raised the buckets after levelling the soil with hydraulic excavators or small pressure charges.

/202/

The work was completed on 20 July 1968 and on 21 July 1968 the foundation area was accepted for installation of the equipment for levelling the bed beneath the TPP block. The foundation area levels were uniform to the required tolerances of ± 15 centimeters. Determination of the bottom levels during the final stage was carried out with the aid of a special depth gauge (section 10-3) over a 2 x 2 meter grid. In order to eliminate the possibility of having missed high spots in the intervals between the points measured, a rough sweep of the foundation area was performed before transfer with the aid of a control gauge and a special frame (section 10-4). During the process of excavation the levels of the foundation area bottom were determined by measurements using a hydrological winch of the "Luga" type. Synchronous observations were carried out at the gauging station in order to determine cut-offs.

Underwater excavation of the foundation area lasted from May 1964 to July 1968, i.e. four years and three months. It cost 630,000 rubles. The total volume excavated (with a slight excess) was 26.3 thousand m^3 . The average cost of excavating 1 m^3 of soil (over the entire period of operations) was 24.3 rubles. The full cost of the operations did not exceed the budget and the length of time the work took, in the final analysis, did not delay installation of the floating block at the site.

Thus, it may be stated that the set task of preparing the underwater foundation area for the experimental structure erected by the floating method was carried out.

The negative aspects of this experiment, however, cannot be ignored. For large volume operations, which would be required for industrial plants, the methods used above cannot be deemed applicable: the required time and cost (the 24.3 instead of 10 rubles per m^3 cost includes general overheads

and planned savings) are excessively high. This is explained by the relatively small volume of excavation and the crowded conditions at this site, as well as by the experimental nature of the operations and their delay during the first stage of construction, when work on installation of the embankment stopped for six months due to exclusively organizational reasons, and subsequently, from November 1965 to March 1966, when work was suspended due to delays in loosening the ground by means of concentrated explosive charges. The general inefficiency of loosening the ground by means of pressure charges must also be acknowledged. It is clear that a completely different effect could have been obtained by loosening the soil with the aid of explosive charges placed into the soil, but special drilling equipment is required for this. To construct powerful TPPs this equipment must be installed not on pontoons but on platforms whose legs would rest on the bottom. This would permit work from this platform, which would be raised above the level of the surging sea, and make it independent of the weather and the level of the tide (as is already being done using floating drilling installations of the Dutch "Hazar" type or the Soviet "Apsheron" type at the Baku Sea oil fields).

b) The Temporary Embankment

The project specified that, taking into account the specifics of the tidal flow, the embankment be constructed in the shortest possible time (3 weeks) and with the minimum weight of stone used to close up the gap (section 3-2).

The indispensibility of complying with the design specifications was necessitated by the fact that the embankment had to be completely dismantled after the TPP block was installed at the site and the connections were made. For this reason the planned construction of the embankment using dump trucks with the formation of a ramp and guarantee of the required surface width was imperative. In violation of the plan, however, dumping of the embankment was carried out without any system whatsoever. The approach ramp to the damming site was not constructed, the bulldozers which were used instead of dump trucks pushed soil into the water indiscriminately, including soft soils and fines, which were immediately washed away and deposited in nearby areas, including the future excavation site. An unsubstantiated explosion to blast out the material only cluttered the bed, even slightly increasing the volume to be excavated. Contrary to the plan, in order to complete the span a 15-ton self-propelled crane was brought in which, while travelling along the bank, loaded up with blocks weighing 5-10 tons and then placed them into the core of the embankment.

Violation of the project plans resulted in the throat of Kislaya Guba being dammed (Fig. 10-3), but made impossible its subsequent dismantlement, using the available machines, within the established period of time. For this reason it was decided to use the "Valmet" floating crane to dismantle the embankment. The high lifting capacity of this crane guaranteed complete dismantlement of the embankment, including the removal of oversized blocks. In order to exclude the appearance of high current speeds, however, it was decided to carry out dismantlement under the protection of the TPP structure, using its ability to regulate the water levels in the basin and, in addition, it was necessary to provide for the crane to be brought into the basin since, with the damming of the site it was cut off from the basin.

/204/



Рис. 10-3. Банкет и губа Кислая.
На переднем плане плавучий кран. Фото
Л. Б. Бернштейна.

Figure 10-3. The temporary embankment and Kislaya Guba. The floating crane is in the foreground. Photographed by L.B. Bernshtein.

The proposed scheme¹ made it possible to bring the crane 1 into the basin and to bring it out to the sea wharf. According to this plan the sheet pile curtain on the west bank 2 is cut off at the 40.50 meter mark. After additional welding and a vertical cut, two panels are formed (Fig. 10-4) weighing approximately 5 tons each. These panels are removed at low water and, when the tide has risen sufficiently (the draft of the crane is 1.50 meters) the crane passes over the cut off sheet pile and closes the gap at low water. The method of removing the crane is similar to that of bringing it in, although it is somewhat complicated by the presence of higher current speeds in the gap when the plates are removed and for this reason the crane is moved out only when there is a small difference in levels and the speeds do not exceed 1 meter per second. The cut-off level is designated such that there is sufficient time to install

¹Proposed and executed under the direction of S.L. Gel'fer, Engineer.

the plates after the crane is removed and, in work above the water level (cut-off level 1.05 meters above the theoretical null depth), to be able to install and secure them with the aid of cable braces. At a design high water level of 42.35 meters, the time required to pass the crane through is two hours and 18 minutes. /205/

It should be noted that the serious nature of this operation elicited well-founded concern from the Rescue and Salvage, Shipraising and Underwater-Technical Operations Organization for the safety of the floating crane and its crew. This made it necessary for the author of the proposal to carefully calculate all parts of the operation, taking into account possible emergency situations. The author's supervision and the Chief Construction Engineer were obliged to accept full responsibility for the safety of the crane and crew.

As planned, on 8 December 1968 the panels were removed by the crane and placed next to the bank. At high water the crane passed over the cutout part of the sheet pile curtain. After the passage of the crane additional earth was dumped around the sheet pile curtain in order to firmly anchor it in the ground after which, on 11 December 1968, the panels were installed and fastened in place. On 12 December 1968, dismantlement of the embankment was begun with the aid of the crane (Fig. 10-5) and by 21 December 1968 the main part of the work was completed. The earth was delivered to the east shore where a power shovel moved it away from the bank. Part of the earth was loaded onto trucks and transported to the dams or to the area for levelling. The crane also removed a large number of oversized rocks without the use of any other machines.

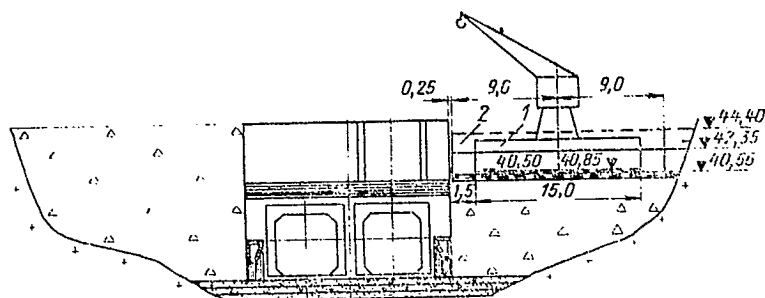


Рис. 10-4. Схема перевода плавучего крана через шпунтовую стенку для разборки банкета в бассейне.

Figure 10-4. Schematic diagram of the movement of the floating crane over the sheet pile curtain in order to dismantle the embankment in the basin.

On 21 December 1968, when only minor clearing still had to be done near the banks and eight oversized rocks had to be removed, the sheet pile curtain was found to be in a dangerous condition. This was a consequence of the delayed start of testing of the TPP structure under critical regimes. Because water was being discharged from Kislaya Guba instead of the levels being equalized, the curtain was periodically under significant (2 meter) head. As a result of the fact that the section of sheet pile near the bank did not closely fit the foundation and the bank, and as a result of the poor quality of the work during the installation of the panel at the west bank, the pressure displaced the lower part of the sheet pile curtain beneath this panel. For this reason the crane was removed from dismantlement of the embankment and, with greater difficulties than during its entrance (because greater differences in level and higher speeds were evident in the gap after dismantlement), the crane was brought out of the basin on 22 December 1968. After the crane was brought out and the bank section was repaired, the sheet pile curtain was re-installed and the connecting dam was completed. All this made it possible, in compliance

/206/

with the schedule of the State Commission for Acceptance of the TPP, to begin operational testing by the end of December.

Thus, it was possible to decrease the volume of excavation, to decrease the period of dismantlement of the embankment from 25 to 14 days, and to significantly lower the cost of constructing and dismantling the embankment (the planned cost was 36 thousand rubles).

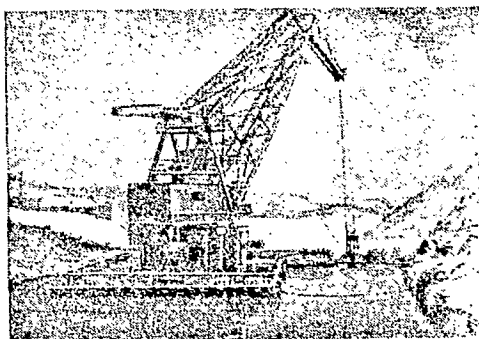


Рис. 10-5. Начало разборки банкета плавучим краном «Валмет». Фото С. Л. Гельфера.

Figure 10-5. Start of dismantlement of the embankment by the "Valmet" floating crane. Photographed by S.L. Gel'fer.

The construction of the embankment, which made it possible to simplify the operations in the foundation area during the construction of the Kislaya Guba TPP, was necessary and expedient only at this site, a special feature of which is the narrow throat, through which pass significant discharges at high speeds (up to 3.5 meters per second) into a large reservoir.

/207/

When powerful TPPs are built which will cut off large bays of the sea, and where the speeds are significantly smaller, the construction of such an embankment will not be necessary.

10-2. Special Features of the Underwater Engineering Operations when the Floating Method of TPP Construction is Employed.

The successful construction of a hydroelectric power structure using the floating method depends on the execution of underwater engineering operations which include: underwater preparation of the bed and joining of the individual floating blocks to each other [14] and to the bed, and with the grouted cut-off in particular [15]. The operation qualities and useful life of the structure depend on the design decisions and the successful execution of these operations. These operations and the equipment for carrying them out are closely interrelated with the operations of bringing the blocks to the site, bracing them over the bed and installing them onto the bed. They are also tied in (especially the levelling of the bed) with the design of the floating blocks. The thickness of the bottom and the system and spacing of bulkheads in the floating block affect the selection of the method of levelling the bed. The effect on the structure of the pressure of the water which, in this case, periodically changes in magnitude and, up to 4 times per day, in direction, may under certain conditions result in scouring of the soil of the bed at the contact with the bottom of the block [21, p. 58]. This affects the selection of both the percolation profile of the structure and the type of levelling of the bed.

A facilitating circumstance in the selection of the type of levelling of the bed beneath the Kislaya Guba TPP structure, which consists of a single floating block without a grouted cut-off beneath it, was the installation of an anti-piping frame consisting of three longitudinal and seven transverse metal blades 200 millimeters high and 11 millimeters thick. The possibility of cementing the contact between the bottom of the block and the bed was also taken into account, for which reason a system of pipes, extending to the upper levels of the block, was incorporated within the vertical limits of the blades. Under the conditions of this experimental structure, however, cementing would have the unfavorable consequence of putting out of action the instruments installed beneath the block, including: piezometers, piezodynamometers and ground dynamometers, which are designed to investigate the static operation of the block structure. For this reason cementing may be carried out only in the event of the arising of circumstances threatening the safety of the structure. /208/

The above set of interrelated problems resulted in the selection, at the Kislaya Guba TPP, of a very precise levelling of the bed, permitting a deviation of the levels of the levelled surface of the bed within the limits of ± 3 centimeters from the horizontal plane at the 30.65 meter mark.

In order to carry out the very precise levelling of the underwater bed, various types of equipment were developed for manual and mechanized levelling (section 10-4). The first of these was a 22.5-meter long control depth gauge, and the second was a pan hopper, consisting of two frameworks with a span of 20 meters. The gauge and hopper are moved along guides which are assembled out of seven 6.5-meter long sections in the first instance, and 8-meter long sections in the second. In order to ensure precise levelling of the underwater bed to within ± 3 centimeters it was necessary to limit the sag of the gauge and the hopper, the rises and falls of the

guides and their component parts, and also the deviation from the vertical of the equipment for transmitting the level marks from above and below the vertical of the equipment for transmitting the level marks from above and below the water.

This necessitated the selection of a method for transmitting the levels which would ensure a precision of ± 5 millimeters. This precision of the transmission of levels underwater and back to the surface is not achieved by the water levelling method which is widely used in underwater engineering operations. This method is based on the use of the sea level as a reference plane whose level is determined from hydrometeorological station or post data. The elevation of an underwater point is calculated from the known water level and the measured depth. The following instruments are usually used for measuring depths: a sounding rod which is precise to ± 5 centimeters, a hand lead which is precise to $\pm 10-20$ centimeters, and a fish lead whose precision is higher than that of the hand lead but lower than that of the sounding rod. In addition, water levelling under the conditions of the water surface at the Kislaya Guba TPP site would have been unproductive as the result of waves caused by the wind and due to the tidal fluctuations of the sea level.

The required precision of transmission of the elevations underwater and back up to the surface is easily accomplished by the method of geometric levelling by means of which the elevation is transmitted, with the aid of a horizontal directional ray, from a bench mark on the shore directly to the point being levelled underwater. In order to do this the levelling rod is attached to the upper, dry part of the depth gauge, which is placed on the underwater point since the depth of the water in the foundation area usually exceeds the length of the rod and hampers its installation in a vertical position. In order to use this method a new depth gauge (section /209/

10-3) was developed which is stable during the transmission of elevations from the surface of the tidal sea at the construction site.

The required precision of levelling of the underwater foundation area beneath the floating structure of the tidal electric power plant was set within rough levelling limits permitting level deviations of ± 15 centimeters from the horizontal plane at the 30.15 meter mark. The need for control, especially of the upper limit, and the incomplete excavation of the earth in the foundation area made it necessary to include a stiff sweep in order to accomplish levelling of the underwater bed to exacting standards (section 10-4).

The underwater activities of the divers installing the sand and gravel bed beneath the floating TPP structure had to be supported by surface operations; particularly the transport and delivery of instruments, metal structures, concrete blocks and earth to the underwater working site of the divers. In order to accomplish this, communications are necessary between the divers and persons working above water: on the shore, at various structures and on floating equipment. However, telephone communications, which are widely used at the present time, do not permit efficient and safe conduct of the construction process in the absence of commonly visible reference points. This complicates the execution of even the most simple operations, such as providing a diver with small and light objects (tools, measuring instruments, etc) which can be lowered on guide lines or off to the side of a diver until he sees them. Subsequently these objects are lowered on the orders of the diver.

When divers wear equipment with which they exhale into the water, or when they wear ventilated suits, it is possible to judge the underwater location of a diver by the air bubbles which rise from him to the surface. For industrial purposes this method of determining the diver's whereabouts cannot be deemed satisfactory. The water current and the movements of the diver result in the bubbles reaching the surface at a point other than above the diver. In addition, the bubbles appear as separate groups over certain intervals of time. When such a method is used to determine the location of a diver, objects and loads are delivered to him slowly, and when hoists are used they must be frequently stopped in order to enable the diver to see the load. This lowers labor productivity.

/210/

The above circumstances were the reason we developed a new apparatus for projecting the underwater position of points onto the water surface and back down again with simultaneous fixation of their planar position on the surface of the sea and on the bottom underwater (section 10-3) with a precision sufficient for the purposes of carrying out the work and independent of the tidal phase.

The system of instruments and installations created for carrying out the underwater engineering operations under the specific conditions of the TPP construction area made it possible to successfully carry out the following operations: to check the levelling accuracy of the bottom of the foundation pit and to detect incomplete excavation, to check the pouring and very precise levelling of the sand and gravel bed, transmission with the aid of the levelling instrument of elevations underwater and back up to the surface, and also projection to the surface of the underwater location of objects regardless of the phase of the tide. This makes it possible to count on the possibility of successfully carrying out underwater engineering operations using the floating cofferdamless method of construction for powerful industrial TPPs.

10-3. Apparatus for Transmitting Vertical and Planar Positions of Points Underwater and Back Up to the Surface

In carrying out underwater engineering operations the transmission of vertical elevations underwater and back to the surface with a high degree of accuracy is accomplished with the aid of a depth gauge. The depth gauge is usually constructed using a wooden pole whose lower end is inserted into a metal base which cancels out the excess bouyancy of the pole [47, p. 107], or it is constructed out of aluminum pipes, connected to each other with spring locks, and whose lower section is equipped with a stop block and is filled with shot, while the upper section is closed with a lid. /211/

The depth gauge is successfully used at water construction sites at which the water level is constant during the period of construction and at which there is a narrow range of depths. This range may be extended with the aid of detachable sections in the upper part of the depth gauge. It cannot be used, however, under the water conditions at the site of a tidal power plant where the water level changes depending on the phase of the tide, where the detachable sections would constantly have to be manipulated from the floating construction equipment, and where the relocation and installation of the depth gauge at the levelling points is carried out primarily by divers.

In preparing the bed beneath the floating structure of the Kislaya Guba TPP the range of working depths, dictated by high spots in the various parts of the bed and the structures for levelling it, was 8.05-8.55 meters relative to the low water level. Taking into account the tidal fluctuations of sea level the range was as high as 8.05-13.25 meters. For this reason the rated range of working depths for the depth gauge was taken to be 7.5-14.0 meters, which covered all of the main underwater points being levelled regardless of the phase of the tide. This determined the depth gauge length of 15.2 meters.

In the given range of depths the above mentioned depth gauge would be inoperative as a result of its inadequate stability resulting from the impractical distribution of the forces of weight and bouyancy along the depth gauge as well as their large absolute magnitudes. When a depth gauge with a pole diameter of 10 centimeters is moved from a levelling point located at a depth of 7.5 (14.0)meters, at which the depth gauge has neutral bouyancy, to a point at a depth of 14.0 (7.5)meters there is a change in the weight displacement (excess weight) of 51 kilograms. In the first instance the diver, who weighs 6-8 kilograms underwater, would be torn away from the bottom and lifted up approximately 5.5 meters; in the second instance the added weight of the instrument being moved would exceed the permissible weight and would quickly result in a physical strain on the diver.

The shortcomings in the depth gauge described above, which would manifest themselves during operation over a large range of depths, were the reason we developed a new type of depth gauge during the design of the TPP [16]. This depth gauge was built at the experimental mechanical repair shop of the All-Union Planning, Surveying and Scientific Research Institute. The depth gauge (Fig. 10-6) consists of a bouyancy chamber, to the bottom of which a weight is attached with the aid of a rod (Fig. 10-7,a), and to whose top two levelling rods are attached (Fig. 10-7,b). The attached weight is 157 kilograms with a total depth gauge weight of 394 kilograms, while bouyancy is achieved with a weight displacement of 362 kilograms. This resulted in stable floating of the body with a negligibly small water line area (downward movement of the center of gravity of the body beneath its center of bouyancy, depending on the depth to which the depth gauge was submerged, was 1.89-1.95 meters). In order to decrease

/212/

the heeling moment, due to the wind and the slight current resulting from the percolation of the water through the temporary embankment closing off the throat of the inlet, the depth gauge was constructed in such a way as to minimize the sail effect both above and below the water. The rods

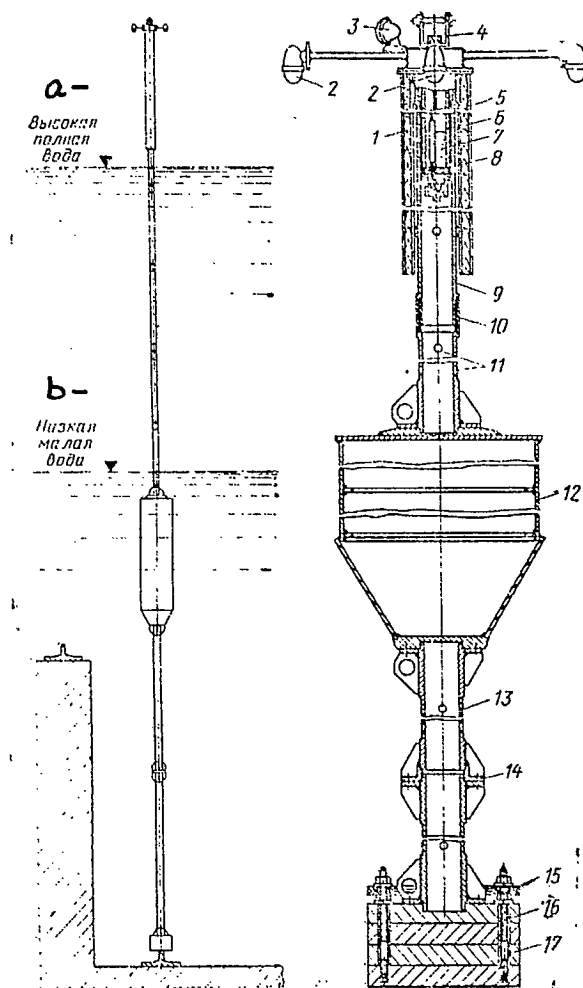


Рис. 10-6. Боковой вид и продольный разрез футштока.

1 — нивелирная рейка; 2 — фара с контрольной лампочкой; 3 — фара с сигнальной лампочкой; 4 — вертлюг; 5 — винилхлоридный стакан; 6 — отвес; 7 — масло; 8 — контактор; 9 — труба верхней штанги; 10 — ниппельное соединение труб; 11 — отверстие; 12 — плавучесть; 13 — труба нижней штанги; 14 — фланцевое соединение труб; 15 — сектор; 16 — шпилька; 17 — диск груза.

Figure 10-6. Side view and longitudinal section of the depth gauge.

1 - levelling rod; 2 - lamp with control bulb; 3 - lamp with signal bulb; 4 - swivel; 5 - rigid polyvinyl chloride sleeve; 6 - plumb; 7 - oil; 8 - contactor; 9 - upper rod pipe; 10 - nipple pipe fitting; 11 - opening; 12 - buoyancy chamber; 13 - lower rod pipe; 14 - flange pipe joint; 15 - segment; 16 - pin; 17 - weight disc; a - higher high water; b - lower low water.

were constructed out of thin-walled pipes containing a system of openings for the entrance of water. The pipes of the lower rod were connected by means of flanges while those of the upper rod were connected with the aid of nipple thread. The increase in the weight displacement (excess weight) of the upper rod during submergence (raising) of the depth gauge within the working range of depths is 0.7 kilograms per meter. This makes it possible for a worker in a small boat, or a diver, to move and install the depth gauge. The bottom weight of the depth gauge consists of four discs 350 millimeters in diameter, as per the engineering specifications [41, p. 19]. The discs are connected by means of pins onto whose upper ends 15 segments may be added with a total weight of 10 kilograms. In order to eliminate unnecessary surfacings of the divers in search of the depth gauge, at high water fourteen segments were used. As a result, the depth gauge was always in contact with the bottom, exerting a pressure of approximately 8.0-3.5 kgf depending on the depth of the water, and remaining at its last measurement site. /214/



Рис. 10-7. Внешний вид футштока. Фото В. Г. Гаврилова.
а -- нижняя часть; б -- верхняя часть.

Figure 10-7. External view of the depth gauge. Photographed by A.G. Gavrilov.

a - lower part; **b** - upper part.

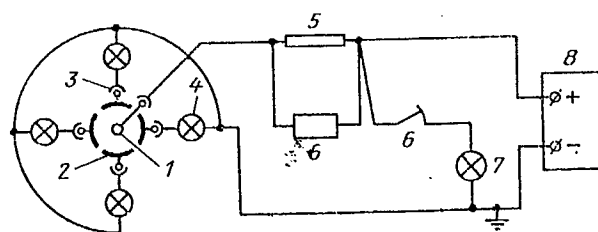


Рис. 10-8. Схема электрических соединений футштока.
 1 — отвес; 2 — сектор контактора; 3 — контакт разъема; 4 — контрольная лампочка; 5 — сопротивление ПЭ-20 5 ом; 6 — реле, среднее напряжение 4—6 в; 7 — сигнальная лампочка; 8 — аккумулятор, напряжение 12 в.

Figure 10-8. Schematic diagram of the electrical connections of the depth gauge.

1 - plumb; 2 - contactor sector; 3 - disconnect contact; 4 - control bulb; 5 - 5 ohm PE-20 resistor; 6 - relay, average voltage 4-6 volts; 7 - signal bulb; 8 - 12 volt storage battery.

An apparatus was built into the upper part of the depth gauge to control its vertical position. The apparatus includes a plumb and a contactor consisting of four segments mounted in a rigid polyvinyl chloride sleeve which is filled with oil and located within the rod. A downward pointing control bulb is installed on a bracket over each segment, and over the levelling pole itself there is a signal lamp pointed toward the geodesist. When the depth gauge deviates from the vertical by more than $1^{\circ}30'$, which corresponds to a 5-millimeter calculation error in terms of the levelling rod, the plumb touches one of the four segments. At that instance the electric circuit is completed (Fig. 10-8), and the control lamp lights up over the segment which the plumb touches and the signal lamp goes out, warning the geodesist not to make erroneous calculations using the levelling pole. Having seen that a control lamp has lit up, the worker in the boat smoothly bends the depth gauge in the direction of the unlit control lamp on the opposite side. As soon as the depth gauge attains a true vertical position the control bulb goes out and the signal

bulb lights up. The current for the electric part of the apparatus is designed to be delivered through a cable strung to the boat where a storage battery is located.

/215/

The transmission of elevations underwater and back up to the surface with the aid of a depth gauge at the Kislaya Guba TPP construction site was accomplished by the following method. A diver, travelling along the bottom of the foundation area, or a worker in a boat, would move the depth gauge and install it at the point to be levelled. Under the influence of the righting moment the depth gauge automatically assumed a vertical position. Using a levelling apparatus located on shore, the elevation from the depth gauge rod was recorded. The elevation of the point being levelled was calculated from the known elevation of the levelling apparatus' horizon and the distance from the heel of the rod attached to it (13.01 meters).

Even though the work was carried out in windy weather there was no need for a special apparatus to control the vertical stability of the depth gauge [19]. The depth gauge, which possesses a high righting moment, remained in a vertical position or went through small oscillations. The position of the depth gauge between the two mutually perpendicular planes was easily controlled in terms of the vertical and horizontal lines of the levelling grid.

In carrying out underwater engineering operations the transmission of the position of the points underwater and back up to the surface, and their subsequent marking on the bottom and on the surface of the water, is accomplished with the aid of a buoy whose signal float is attached by a line to an anchor embedded at the assigned point on the bottom. This apparatus, which is widely used at building sites where the water level is constant during construction and where the range of depths is narrow, is

unsuitable for work in a TPP foundation area where the water level changes depending on the phase of the tide. If the length of the line is made equal to a depth of 8.05 (13.25) meters, which corresponds to low (high) water, then during high (low) water the buoy will be submerged (swept 10 meters to the side).

At the Kislaya Guba TPP construction site, regardless of the phase of the tide, the planned position of the points was transmitted underwater and projected back up to the surface of the sea with the aid of a newly designed buoy [17], manufactured by the Central Mechanical and Maintenance Shops of the Northern Administration for Hydraulic Engineering Construction. The buoy (Fig. 10-9) consists of a signal float (1) with a weight displacement of 64.9 kilograms, floating on the surface of the water. The signal float is connected by a rope (2) with a counterweight (4), weighing 30.7 kilograms underwater. The rope passes through the central opening of a stabilizer float (3) with a weight displacement of 82.5 kilograms. The stabilizer float is held 0.8 meters below the low-water level by means of a rope (5), which passes through the central opening in the counterweight and is attached to the anchor (6), which is embedded at the assigned point on the bottom. /216/

In carrying out the underwater engineering operations the buoy described above was an underwater and surface marker which could be easily set out at the assigned point on the bottom or surface of the water by persons working either under the water or on its surface.

The high efficiency of the new buoy under the conditions of the Kislaya Guba TPP construction site allows us to expect that it will be widely used in measuring depths from floating structures, in examining the bottom of an expanse of water or a foundation area by divers, in the transfer of full-scale layouts of structures, and in many other instances

when assigned locations, axes or areas have to be marked and fixed simultaneously on the bottom and on the water surface for the subsequent execution of underwater and surface operations at these locations. -

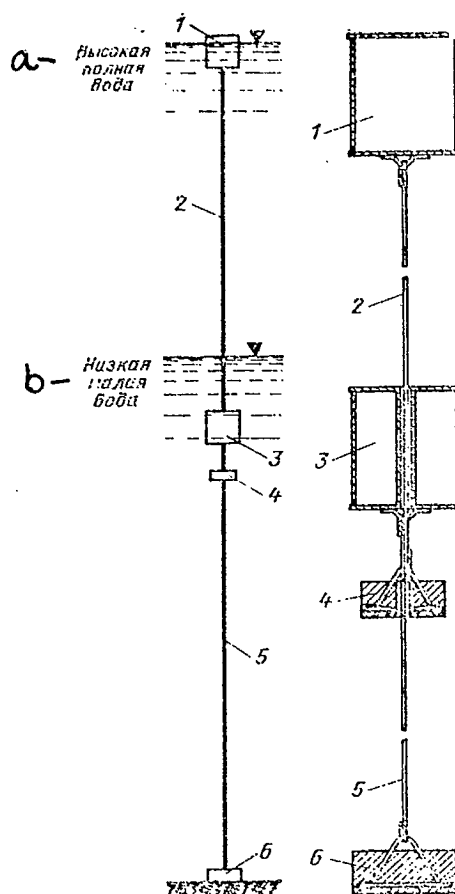


Рис. 10-9. Боковой вид и продольный разрез буя для строительной акватории на приливном море.

Figure 10-9. Side view and longitudinal cross section of a buoy for a construction site in tidal seas.

a) higher high water; b) lower low water.

10-4. An Apparatus for Levelling the Bed

Soviet and foreign practice of carrying out underwater engineering operations includes a series of technological methods for very precise levelling of the bed when the method of construction does not make use of a cofferdam. These include, for example, an MPP-1 mechanical underwater leveller and a specialized SPU-1 floating installation used for levelling underwater rock beds at sites of the Baltic Marine Administration for Hydraulic Engineering Construction of the Ministry of Transportation Construction, USSR, or a planing scraper used in the construction of the transport tunnel beneath the Kiel Canal near the city of Rendsburg, and many others. However, as will be described below, these apparatus could not be used at the Kislaya Guba TPP construction site and new equipment had to be developed.

At the Kislaya Guba construction site there are fluctuations in water level caused by regular semidiurnal tides 1-4 meters high, and waves caused by wind may also appear. This impeded the use of equipment for which the water level is the primary working plane. The width of the throat of the inlet in the area of the floating TPP structure at the time the bed was being prepared was 50-70 meters at the water line at the higher high water and lower low water levels. These conditions made it impossible to use apparatus, including guiding equipment, equipment spanning both banks, and floating equipment, located outside of the perimeter of the bed being levelled. The bedrocks outside of the perimeter of the TPP block are covered with a thin layer of consolidated marine deposits, including boulders, and in some places they extend into the foundation area. This prevented the use of equipment held in place on piles driven into the bottom of the foundation area. Included among the peculiarities under discussion is the composition of the sand-gravel soil of the bed, with an

individual particle size range of 0.1-50 millimeters, which may separate into layers during dumping and which becomes turbid when it is moved underwater, resulting in decreased visibility. In addition to the above factors, also taken into account were the small area of the bed being levelled (950 m²), the limited periods of time set aside for levelling the bed and building the structure, as well as the availability of devices, machines and floating equipment to the subcontracting organizations (the Expeditionary Unit for Rescue and Salvage, Shipraising and Underwater-Technical Operations of the Murmansk Marine Shipping Administration, which was entrusted with the job of pouring and levelling the bed).

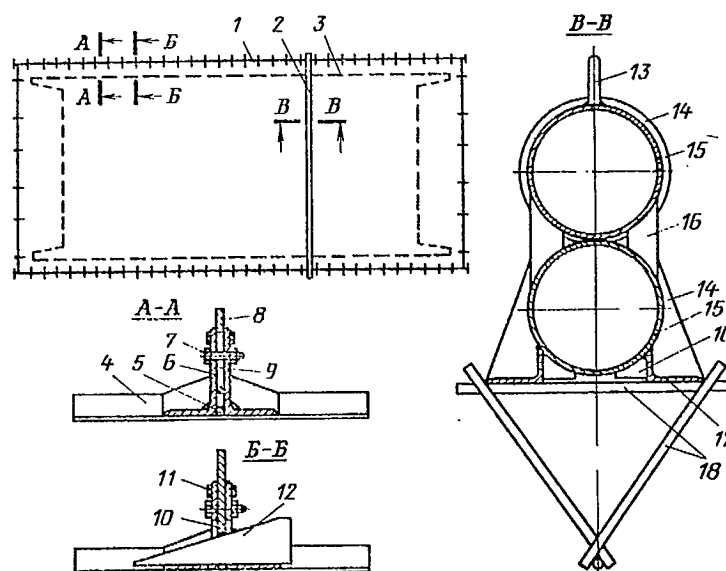


Рис. 10-10. План и поперечные разрезы устройства для ровнения подводной постели водолазами.

1 — направляющая рама; 2 — контрольная рейка; 3 — контур наплавного здания ПЭС; 4 — шпала; 5 — вкладыш; 6 — направляющий уголок; 7 — болт; 8 — опорная полоса; 9 — прорез для болта; 10 — окно для клина; 11 — накладка над окном; 12 — клин; 13 — монтажная петля; 14 — торцевая заглушка трубы; 15 — труба; 16 — фасонка; 17 — контрольный уголок; 18 — съемный трал.

Figure 10-10. Plan and transverse sections of the apparatus for levelling the underwater bed by the divers.

1 - guiding frame; 2 - control rod; 3 - perimeter of the floating TPP structure; 4 - tie; 5 - insert; 6 - angle guides; 7 - bolt; 8 - supporting bar; 9 - slot for bolt; 10 - opening for wedge; 11 - cover plate over opening; 12 - wedge; 13 - assembly loop; 14 - pipe end cap; 15 - pipe; 16 - fitting; 17 - control angle; 18 - detachable sweep.

The apparatus for very precise levelling of the bed, which was /218/ designed by us [19] and built at the Central Mechanical Repair Shops of the Northern Administration for Hydraulic Engineering Construction (Fig. 10-10), includes a guiding frame 45.5 meters long and 21.0 meters wide. The size of the frame was governed by the dimensions of the block and the levelled bed margins outside of the perimeter of the block. In terms of the bottom plate the block is 36.0 x 18.3 meters, while at a height of 1.5 meters from the bottom the length of the block is 42.5 meters as a result of its projections. The frame is assembled with the aid of bolts out of 14 longitudinal and 6 transverse sections. Each longitudinal section consists of two angles whose horizontal wings are welded to the ties and between whose vertical wings, which are held together with bolts, a supporting bar is inserted with slots for the bolts. In the vertical wings of the angles there are openings for wedges, which make it possible to regulate the vertical position of the supporting bar within the limits of 90 millimeters. Each transverse section consists of an angle with two ties welded /219/ to it.

During the process of underwater assembly of the guiding frame it was suggested by the author's supervision [19] to weld two rods of reinforcing steel 70-120 millimeters long and in the shape of a T to the heads of the bolts which would be in a horizontal position; it was suggested that the heads of the bolts which would be in a vertical position should be welded to the frame sections before they were lowered underwater. This speeded up the underwater assembly of the guiding frame.

A 22.5-meter long control rod is placed on top of the supporting bars of the longitudinal sides of the frame. The technical specifications [41, p. 20] recommend a control rod length of 5-6 meters. The permitted deviation from the technical specifications is based on the fact that their recommendations are for a rod manufactured out of a narrow gauge rail, and which thus limit the weight of the rod underwater to 45-60 kilograms and also limit its sag. The adopted length of the control rod made it possible to avoid the undesirable installation of longitudinal guides within the sand-gravel bed and to decrease the volume of underwater operations for their installation by 2.5 times.

According to plan, the control rod consisted of two drilling pipes 273 millimeters in diameter and with 7 millimeter thick walls, connected by means of a sleeve. The ends of the pipes were closed off in order to create buoyancy, which decreased the weight of the rod underwater to 50 kilograms. Angles were welded to the lower part of the pipe along both sides of the rod. These angles are the bottom guides, while the pipe itself is the guide along the sides for the shovel, which the diver moves and with which he cuts away the excess earth which has been dumped higher than the horizontal wing of the control angle. A detachable rigid sweep suspension member was designed for the rod (Fig. 10-10) consisting of a framework 20 meters long and 38 centimeters high. Deviations which were permitted from the design during construction of the rod resulted in an increase of its weight. In order to rectify the situation, and on the basis of the pipes available at the construction site, we proposed that a 22-meter long pipe like that of the control rod be welded to it. In the end, the weight of the rod underwater was adjusted to the design weight but its drag was significantly increased, which made repositioning of the rod more difficult but did not hinder the successful realization of the project.

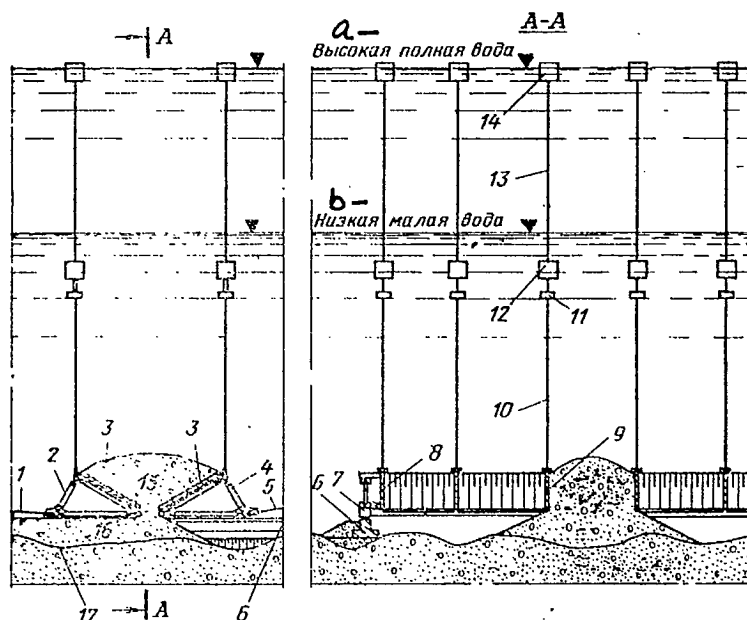


Рис. 10-11. Поперечный и продольный разрезы механизированного устройства для образования подводных постелей.

1 — канат лебедки холостого перемещения; 2 — ведомая ферма; 3 — обшивка (стенка бункера); 4 — ведущая ферма; 5 — канат лебедки рабочего перемещения; 6 — опорная балка направляющей; 7 — ходовое устройство; 8 — торцевая стенка бункера; 9 — поперечная переборка бункера; 10 — трос поплавка-стабилизатора; 11 — противирос; 12 — поплавок-стабилизатор; 13 — трос поплавка-указателя; 14 — поплавок-указатель; 15 — песчано-гравийный грунт в секции бункера; 16 — отсыпанная и выровненная полоса постели; 17 — дно котлована.

Figure 10-11. Transverse and longitudinal sections of the mechanized apparatus for forming the underwater beds.

1 - rope of winch for idle movement; 2 - following framework; 3 - sheathing (hopper wall); 4 - leading framework; 5 - rope of the driving movement winch; 6 - guide support bar; 7 - travelling mechanism; 8 - end wall of hopper; 9 - transverse partition of bunker; 10 - stabilizer float cable; 11 - counterweight; 12 - stabilizer float; 13 - signal float cable; 14 - signal float; 15 - sand and gravel soil in a hopper section; 16 - dumped and levelled strip of the bed; 17 - bottom of the foundation area; a - higher high water; b - lower low water.

In the course of further design work a mechanized apparatus [18] was developed for forming the underwater bed beneath the floating TPP structure under the conditions at Kislaya Guba. The apparatus (Fig. 10-11) consists of a leading and following frameworks of triangular cross section, with sheathed surfaces facing the longitudinal axis of the trough-shaped hopper formed by them. The 20-meter span of the hopper is determined by the 18.3 meter width of the floating TPP structure, by the precision of

installation on the bed, and by the free berms which are very carefully levelled on both sides of the structure. The hopper is 1.15 meters high, 4.0 meters wide at the top input area, and with a 0.5-meter wide output opening. In order to decrease the sag of the hopper the frameworks were welded out of pipes with hermetically sealed ends of all chords and the hopper was divided by transverse partitions into eight sections. The lower edges of the partitions were raised over the output opening of the hopper with the aim of spanning adjacent dumped and levelled strips. Power linkage of the framework chords is achieved through the connection of the transverse partitions. /221/

The frameworks are equipped with double-flanged running wheels for moving the hopper along the guides, which are made out of rails attached to supporting beams. The running wheels are installed with play in order to decrease the designed precision of installation of the guides to ± 3 cm. The height of the rail heads is chosen in such a way that the lower edges of the framework chords, which level the bed, are located higher than the horizontal levelling plane by a distance equal to the permissible upper deviation of the bed elevations. Buoys are attached to the upper chords of the frameworks in order to show the underwater position of the hopper sections on the surface. The operational movement of the hopper is accomplished by two mechanically synchronized winches with a tractive force of 3 ton-force, the ropes from which are symmetrically attached to the lower chord of the leading framework. No-load movement of the hopper is provided for by synchronized winches with a tractive force of 0.5 ton-force installed on the scow. The ropes from these winches are attached to the lower chord of the leading framework.

The dumping and very careful levelling of the underwater bed with the aid of the equipment described above is accomplished by the following method. The trough-shaped hopper is installed in the starting position in front of the bed. The floating crane, which is equipped with a 0.5 m^3 grab bucket, loads 12 buckets of sand and gravel soil into the third and sixth sections of the hopper. At this time the frameworks sag 5 centimeters. The power winches move the hopper forward. The soil is dumped from the hopper and the levelling edge of the lower chord of the leading framework cuts it down to the levelling height of the bed. The distance over which the hopper is moved each time is determined by the underwater volume and thickness of the dumped layer and is specified by the technological chart. Two strips of the bed are prepared in this manner. The final position of the hopper is determined by the distance of the signal floats to the previously established site, and is controlled by the appearance of markings on the winch drum cables, after which the no-load winches move the empty hopper back to the starting position and the cycle is repeated. During subsequent runs the second and seventh sections of the hopper are filled, then the fifth, then the fourth, and finally the first and eighth. This sequence was chosen so that the strips of bed would be formed first beneath the sections having greatest sag when the frameworks are filled. This decreases the tractive effort because during subsequent runs the soil from previously completed strips is not touched.

/222/

The above apparatus and method make it possible to carry out dumping and very precise levelling of a sand and gravel bed with deviations of its surface elevations of ± 3 centimeters from the levelling plane. It is possible to increase the precision of levelling by decreasing the sag of the frameworks or by carrying out control levelling, after formation of

the bed, during which the hopper sections would be filled by the above mentioned method but they would be loaded with a smaller volume of soil. By means of the appropriate structural configuration of the bottom chords of the frameworks or by means of installing blade graders on the apparatus, it can remove drift from the surface of the bed before the floating block is lowered onto it. This apparatus eliminates the inefficient manual work by the divers of levelling the bed and shovelling soil, which would result in layering, which is undesirable under the conditions of a hydroelectric power plant under pressure.

10-5. Underwater Construction of the Bed beneath the Floating TPP Structure

The underwater engineering work on the sand and gravel bed beneath the floating TPP structure in the throat of Kislaya Guba was carried out by teams from the two diving boats "Vodolaz-8" and "VM-80" and from the "Valmet" floating crane, which has a lifting capacity of 10 ton-force.

The divers worked in two shifts. The planned positions of the outer corners of the guiding frame were transmitted underwater with the aid of a layout sounding lead lowered at definite points along a cable which was stretched across the throat of the inlet over the north and south transverse sides of the frame. A diver marked the transmitted positions by driving posts into the bottom from which lines were stretched.

/223/

The guiding frame was installed by the divers onto 60 x 25 x 20 centimeter concrete blocks. The divers began installation of the blocks from the northwest corner post, moving along the longitudinal line and placing them from the side of the foundation pit right up against the line. The distance between blocks was measured with a preset gauge rod. In order to determine the vertical position of the block the divers placed the depth gauge on top of it. The depth gauge measurement was established with the

aid of the on-shore levelling instrument and the divers were informed by telephone how much to raise or lower the block. Raising the elevation was accomplished by turning the block on edge, stacking the blocks on top of each other or placing bricks or stones beneath the block. Lowering the elevation was accomplished by replacing the block with bricks or by placing the block deeper into the bottom. The block was considered to be in place with respect to elevation if its top was between the 30.40 and 30.48 meter marks. The blocks were delivered underwater by the floating crane, during which time the diver and crane operator were in communication by telephone.

When the two end blocks, over which the sections had to be joined, were installed, the first longitudinal section was delivered to the diver. Prior to this, on board the floating crane where the sections were laid out and using the known elevations of the end blocks, the supporting bar of the section was installed and tightened in place in such a way that, after its installation on the blocks, its upper edge would be at the 30.68 meter mark. After installation the middle block beneath the section and aligning the ends of the section ties with the blocks, the diver released the straps. Then the diver placed the end block under the second section, which was installed in the manner described above, and bolted it to the first section. Installation of the sections was complicated by incomplete excavation of the earth along the perimeter of the frame during preparation of the foundation area. Its removal significantly lengthened the time required to install and assemble the frame underwater. After the frame was assembled the height of all supporting rods was checked, some of them were adjusted, and the bolts were tightened. Thus the upper edges of the supporting bars of the guiding frame were installed at the 30.68 meter mark with downward deviations of not more than 5 millimeters. This position

was maintained by the supporting bars without additional adjustment throughout the entire bed levelling period. In order to mark the underwater position of the frame, buoys were attached to its corners.

A rigid sweep was attached to the bottom of the control rod and, in order not to increase its weight, two floats were attached. The rod was lowered onto the support bars of the longitudinal guides on the north transverse side of the frame. Sweeps, carried out by two divers during one shift, revealed inadequate excavation of the earth of 5-12 centimeters above the top 15-centimeter limit for levelling accuracy of the bottom of the foundation area. The earth which had been inadequately excavated was removed as it was found, with the exception of one place where a large boulder was found. The sweep and floats were removed at the south end of the frame. /224/

Buoys were attached at the ends of the control rod, between whose signal floats a line was stretched with floats threaded onto it. Buoys were placed in front of the line along the longitudinal sides of the frame at a distance from the control rod such that, within the area designated by the line and the front signal floats, all of the earth delivered by one scow with a carrying capacity of 100 ton-force could be dumped, thus obtaining an average bed thickness of 50 centimeters. The crane operator carried out the dumping operation, trying to evenly distribute the earth over the given area, since this eased subsequent levelling of the bed by the divers. As the bed was dumped and levelled the divers moved the forward signal floats and the control rod, which moved the line on the surface of the water. Subsequently the earth was dumped using only four signal buoys, two of which were attached to the control rod and two which were moved in front of it. Additional dumping at individual locations was carried out by the crane operator upon the instructions of the divers.

The sand and gravel mixture for the bed was prepared at the Ura-Guba quarry, where fractions larger than 50 millimeters were removed. The sand and gravel mixture was delivered by dump trucks to the wharf and reloaded onto scows which were towed by passenger and freight launches to Kislaya Guba.

In order to prevent layering of the sand and gravel mixture during dumping of the bed the All-Union Scientific Research Institute of Hydraulic Engineering recommended that the 2 m³ capacity grab bucket be opened right at the bottom or at a distance of not more than 0.5 meters from the bottom. When this recommendation was tested under working conditions it was established that when a filled bucket is lowered underwater the sand fractions are washed out and spread over the foundation area [25]. All of the following contributed to this: movement of the water at the surface caused by wind action and percolation of water through the temporary embankment damming the throat of Kislaya Guba; rolling of the soil in the bucket at the instant of its submergence and as it descends underwater, accompanied by a certain spin of the bucket and deviation from the vertical. On the basis of soil samples taken at the quarry, on the scows, and by divers from the bed, it was established that when the bucket is opened at a height of 1.0-1.5 meters from the bottom, 10-15% of the sand fractions are washed away under the influence of the above factors, but there was no significant difference in the distribution of fractions throughout the thickness of the bed. The majority of samples taken from the bed had a density of 1.95-2.08 grams per cm³. The conclusions about the practical uniformity of the mechanical, percolation and piping characteristics of the soil throughout the area and thickness of the bed were drawn on the basis of the above data.

/225/

The divers made one pass with the control rod and levelled the bed very precisely, as indicated by the results of levelling measurements using the depth gauge which was placed at the intersections of a grid with a cell size of 2 x 2 meters. The levels were determined at 242 points located within the perimeter of the frame, 181 of which lay within the perimeter of the TPP block. Discrepancies in the elevation of the surface of the bed beneath the block at 169 points were within the limits of ± 3 centimeters from the horizontal plane at the 30.65 meter level, at 7 points they were within the limits of ± 4 centimeters at 5 points they were within the limits of ± 5 centimeters.

Chapter 11

TOWAGE OF THE FLOATING BLOCK

11-1. Description of the Design Solution

The set of operations involved in towing the block (raising, removal, towing, manoeuvring at the site and submersion onto the foundation) embraces all of the features of the floating method of tidal electric power plant construction (or river hydroelectric power plant construction). The planning and execution of these operations were the *raison d'être* of the entire experiment of building the Kislaya Guba TPP, for which reason they are presented here in as much detail as possible.

/226/

The project was worked out by the All-Union Planning, Surveying and Scientific Research Institute together with the State Planning, Design and Scientific Research Institute of Marine Transportation of the Ministry of the Maritime Fleet (towage), by the Leningrad Marine Planning Scientific Research Institute (excavation) and by the State Institute for Planning in River Transportation¹ (decreasing the draft). The plan called for erection of the building with its long side along the longitudinal axis of the construction area and, after flooding of the construction area, the structure was to be turned 90° to bring it into the starting position to be towed out through the gap (after excavation of the dry land part of the cofferdam with a power shovel). At this time provision was made for a special sand pad because the installation of the block on its original site was found to be impossible due to the protruding bilge pads beneath the bottom and the anti-piping angle bar around the perimeter.

¹The work was carried out under the direction of engineers G.V. Tankhel'son (towage), Yu.N. Novikova (excavation) and B.V. Polyakov (decreasing the draft).

Four manoeuvring T193B winches of the type used in railroad transport, with minor alterations (addition of reverse and idle operation capability), six shore anchors and two 50-ton anchorage buoys were to be installed on the decking of the block in order to turn it and pull it toward the gap. The winch had a tractive effort of 5 ton-force, a cable capacity of 220 meters and a cable take-up speed of 2.6 meters per minute.

The selection of this type of winch was determined by the required calculated tractive effort of 4.8 ton-force and an operating speed of not more than 3 meters per minute due to the hydraulic resistance, the speed of the current and the inertia of the block. The top speed of 3 meters per minute was adopted in view of the conditions of manoeuvring the block while carrying out all of the operations of raising the block, aligning it with the gap, bringing it through the gap and beginning to tow it by sea tug on the tidal wave two hours and thirty minutes before high water. The small size of this winch in comparison with capstans, as well as the availability of the equipment, were factors in this selection.

The following design solutions were provided for in carrying out the towage and placement of the block.

The lower part of the block was divided into a series of waterproof ballast and drainage bays, which made it possible to ballast the block with a high degree of precision while using a minimum of ballast water. In order to take on ballast the block was equipped with sluice valves, with controls at the top for opening them. There was the capability, in case of emergency, of pumping out excess water, by discharging it into the drainage system from most of the ballast bays. The pump system for fire control was also used to take on ballast and the immersion pumps for pumping water out. The system of bays was designed in such a way as

to ensure unsinkability of the block during towage in case of a rupture or the formation of a leak. The block is stable with a metacentric height of $h = 3.97$ meters.

The placement of the block onto the artificial foundation was deemed to be the most important part of the operation. The draft of the block, after removal of the pontoons, bilge pads and cables, was designed to be 8.20 meters, which guaranteed that it would float at low water. Thus, in order to lower the block it was ballasted to 1.5 meters, i.e. to a depth exceeding the margin beneath its bottom. The block was made to touch bottom before the advent of low water. After examination of the position of the block, provision was made for floating it again when the tide came in, in order to reposition it.

The block was ballasted well in advance in order that the bottom of the block would be in a strictly horizontal position. This ensured that the bottom of the block would touch the precisely levelled bed all at once, thus preserving it. This also preserved the integrity of the block by eliminating the possibility of its leaning on a corner or side.

The following plan of preparing for and carrying out the operation was called for by the project.

After flooding of the construction area and positioning of the block in the starting position a bucket dredge was to approach, which would excavate a 62 meter gap (volume of excavation 20 thousand m^3), through which pontoons would be brought in which would decrease the draft of the block from 8.88 meters to 6.21 meters (both figures are clearance drafts which take into account the bilge pads which project 56 centimeters beneath the bottom plate and through which pass the straps of the six pontoons (each with a water displacement of 400 tons) supporting the block.

The towage period was scheduled for the last syzygy in July or (backup) the first and second syzygies in August, because the sea is most calm at this time of the year. The removal of the block was scheduled for the first days of syzygy in order to safely pass the shallow section (the gap and the shoals of Kola Bay before the entrance to the deep channel) and then, after towage over a 60 mile course with the aid of two 1,200 horsepower tugboats and one 500 horsepower tugboat, to bring the block across the shallows of the throat of Kislaya Guba with the last syzygy. Since, during quadrature tides, the block is safe from the danger of unforeseen contact with the bed, it was proposed to free it of the pontoons and bilge pads and to place it on the foundation with the aid of water ballast during the next ebb. If the block was found to be in the planned position after settlement the water ballast would be replaced with sand ballast by the next syzygial tide. /228/

These design studies were substantiated and corrected by careful study of the experience of towing floating docks by the TPP designers at the Kherson City Docks, by inviting specialists from Kherson to Murmansk, by carrying out block placement "rehearsals" at the construction site, by sending a tugboat along the entire towage route, by analysing a film about the towage and installation of floating dam blocks for the Delta Plan in Holland, and by special model and theoretical studies (towing resistance at the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, stability of the slopes of the foundation area and the gap at the All-Union Scientific Research Institute of Hydraulic Engineering).

As a result, it was decided to use three tugboats for towing: two 2,000 horsepower lead tugs and one cant tug moored to the stern pontoon (Fig. 11-1,b). However, in view of the fact that, during model tests, block trim became evident when the speed was increased to 5-6 knots in

shallow waters (at $H/T = 1.2-1.5$), it was decided that the block should be towed in shallow water sections by only one 2,000 horsepower tugboat (Fig. 11-1,b).

Detailed plans were developed for repositioning and removing the block from the construction area at Cape Prityka, for bringing it into the TPP site, including disposition of the mooring rings - anchors along the bank and the anchor blocks along the roadsteads at Cape Prityka and Kislaya Guba, as well as the transfer and winding of the hawsers from the block to these mooring rings and anchors.

/229/

11 - 2. Flooding the Construction Area and Raising the Block

The block of the TPP structure was built in the construction dock (construction area) at Cape Prityka at a depth of 4.3 meters from the low water mark. The construction area was separated from Kola Bay by a small cofferdam built out of the earth excavated from the construction pit (Fig. 11-1). Construction of the floating block was completed on 4 July 1968 and the Worker's Commission, after testing it for watertightness, accepted the construction dock for flooding.

In a departure from the original plans, the sand bed for temporary placement of the block was not located closer to the gap but rather in the empty north part of the foundation area, perpendicular to the platform of the block during its construction (Fig. 11-2).

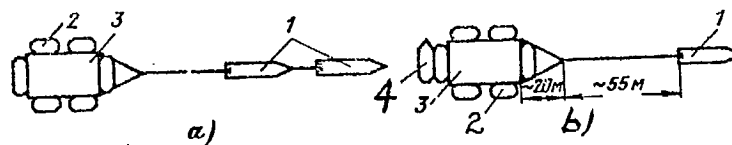


Рис.11-1. Схема буксировки блока.

a — на глубоководном участке; *б* — на мелководном участке; 1 — буксир мощностью до 2 000 л. с.; 2 — понтоны грузоподъемностью 400 тс (6 шт.); 3 — блок ПЭС; 4 — буксир мощностью 350 л. с.

Figure 11-1. Block towage scheme.

a - in a deepwater section; *b* - in a shallow water section; 1 - 2,000 horsepower tugboats; 2 - six pontoons with a lifting capacity of 400 ton-force; 3 - TPP block; 4 - 350 horsepower tugboat.

This change was necessitated by the refusal of the dredge captain to secure the power shovel by means of diverging cables, as had been decided earlier with a representative of the Arkhangel'sk Administration of Sea Routes, and the need, as a consequence of this, to lay the main bearing cable strictly along the axis of the future gap. This axis passes through the standard location on the block construction platform and along its longitudinal axis after the initially planned turn. During cross dredging the cable had to move over the entire width of the gap for a distance of approximately 50 meters. For this reason it was decided to construct the bed for the temporary installation of the block at a new location so that, after flooding of the construction area, the block could be immediately removed and there would be room to lay the bearing cable along the axis of the gap. The temporary bed was constructed with a top elevation of 3.6 meters (from the theoretical null depth) out of sand in order not to damage the anti-piping blade and the bilge pads. For the same reason a narrowed area with extra room was provided for in the area of the bilge pads beneath the side walls. Thus, the width of the bed, equal to 12.5 meters, was smaller than the width of the block, which was 18.3 m, while its length was 30 meters, which resulted (taking into account the projections) in an area of 366 m^2 , or 56% of the area of the bottom plate. The slopes of the embankment were laid at 1:2, and did not change after flooding of the construction area, as shown by sounding measurements. /230/

Due to the apprehension that if ground dynamometers were embedded into the concrete of the bottom plate they would not give readings, it was decided to place them directly into the soil of the underwater bed. In order to train the divers to install these dynamometers and to check their operation, nine dynamometers were installed into the temporary bed prior to flooding and an additional six dynamometers were installed underwater after flooding.

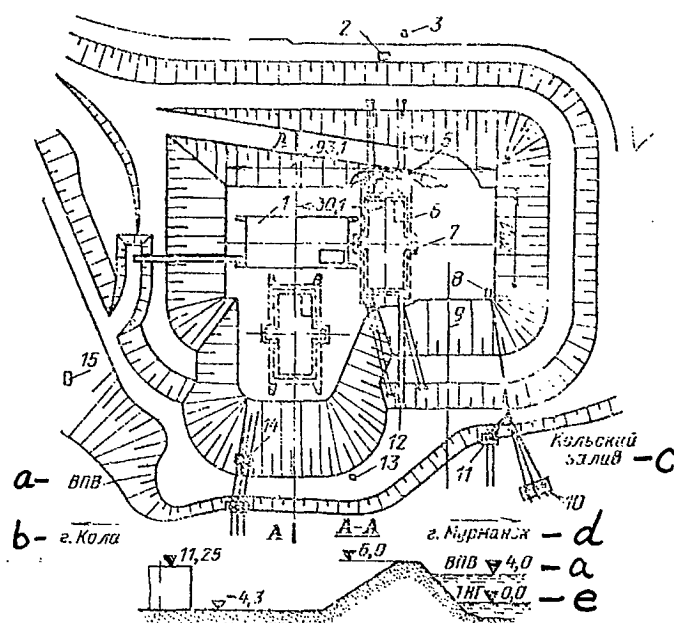


Рис. 11-2. План строительного дока на мысе Притыка.

1 — блок ПЭС до перестановки; 2 — пульт съема показаний с грунтовых динамометров, заложенных во временное основание; 3 — полевой телефон для передачи на пульт отсчетов на рейках блока и данных нивелирования; 4 — цепьковые концы, закрепленные на тросах и выведенные вне пределов затопления; 5 — тросы для понтонов; 6 — блок ПЭС после перестановки; 7 — временная постель под блок; 8 — ряжевый водоприемный колодец; 9 — водомерный пост; 10 — плавучая насосная; 11 — береговая насосная; 12 — понтоновый переход на блок; 13 — полевой пульт наблюдения за рейками на блоке; 14 — насосы (резерв); 15 — запасной пульт управления.

Figure 11-2. Plan of the construction dock at Cape Prityka.

1 - TPP block prior to repositioning; 2 - readout panel of data from the ground dynamometers placed into the temporary foundation; 3 - field telephone for transmitting readings from the block level indicators and the levelling data to the control panel; 4 - hemp ropes attached to cables and brought out beyond the limits of flooding; 5 - cables for the pontoons; 6 - TPP block after repositioning; 7 - temporary bed beneath the block; 8 - crib water well; 9 - gauging station; 10 - floating pumping station; 11 - shore pumping station; 12 - pontoon traverse for block; 13 - field control point for observing gauges on the block; 14 - pumps (standby); 15 - standby control panel; a - higher high water; b - Kola City; c - Kola Bay; d - Murmansk City; e - theoretical null depth.

After rolling the bed smooth, laying the cables for the end pontoons on it, placing the markers for the underwater installation of the dynamometers, and equipping the gauging stations in the dock and in the bay, at 0800 hours on the morning of 5 July the pumps of the dock drainage system were switched off and dismantled and at 1453 hours the pumps bringing in the sea water were switched on. Flooding proceeded at an average rate of 4.3 centimeters per hour and continued until a water depth of 8 meters was attained, after

provision was made for pumping out the construction area and carrying out repair of the bottom plate should the need arise. Since no leaks were discovered, however, flooding of the construction area continued. The outside walls were tested earlier by hosing under pressure and the inner walls were tested by filling the spacings with water.

The block began to float at four o'clock on the morning of 13 July at a water depth of 8.14 meters. At this time the block detached itself from the bed on the shore side and, as water continued to be pumped into the construction area, the block rose with a heel without trim. At 1530 hours, at a water depth of 8.43 meters, when the heel reached 57 centimeters, the first trimming of the block was carried out. This trimming did not immediately show results because the by-pass openings between the spacings in one of the compartments were mistakenly closed off, as a consequence of which the volume was lower than it should have been when the water ballast was taken on. After an additional check of all spacings and an examination of the condition of the block, at 2300 hours of the same day, when the heel reached 90 centimeters (at a water depth of 8.68 meters), a second attempt was made to trim the block during the course of 40 minutes. When the ballast water was taken on this time, at 0010 hours on 14 July, the block floated up with a heel of 21 centimeters by the bow.

At 1200 hours on 14 July the dredge "Dvina" anchored in the roadstead of the construction dock in order to take up the leading cable as stipulated. At this time, however, the water in the construction area had not come up to the planned level due to the unforeseen stoppage of the pumps.

After an additional pump was switched on, by 0955 hours on 15 July, the water level had quickly risen to 9.62 meters and, during the course of four hours, the block was turned 90°, moved, and placed over the temporary bed with a maximum difference of levels at the corners of the foundation of 8 centimeters.

From the data obtained for the draft of the four corners of the floating block, the average draft of the equipped block was determined to be 8.32 meters, which is exactly equal to the design draft. The weight of the equipped block without water ballast was 5,200 tons. Immediately after the block was in position over the temporary bed, at 1400 hours on 15 July, pumping out of the foundation area was begun in order to settle the block onto the temporary bed. It was necessary to settle the block onto the temporary bed in order to prepare for the joining of the construction dock area with the bay and also as a rehearsal for lowering the block onto the permanent bed at the TPP site in Kislaya Guba. As a result of the fact that lowering of the level, as the water was pumped out, showed the recently dumped slopes to be stable, it was decided to speed up the drainage operation by digging through the cofferdam, which was accomplished at 1400 hours on 16 July, and which made it possible to lower the level (depth) of the water in the foundation area from 9.35 meters at 1600 hours to 7.84 meters at 1700 hours, i.e. 2.5 centimeters per minute, or 2-5 times more quickly than under the conditions at Kislaya Guba during ebb, and 25 times more quickly than called for by the project (6 centimeters per hour). The slopes in the gap and the construction area remained stable during this process. The blade touched the bottom two hours later, when the rate of decrease of the level was still 1 centimeter per minute, i.e. was equal to the rate of natural decrease in level of the tide during placement of the block at Kislaya Guba. No significant changes were recorded in the readings of the ground dynamometers. At 1600 hours the block had a heel of 16 centimeters and a trim of 13 centimeters. /232/

The block was installed onto the temporary bed with the aid of range poles on the banks and the operation was carried out with a high degree of precision (deviation from the axis 5-10 centimeters).

After the block was in place on the temporary bed, six pontoons were brought in, one after the other, through the gap, which was sufficiently large by this time. By 4 August these pontoons were rigged to the block on all sides and on 8 August, during the six hours prior to the evening high water, a general blow-out of all the pontoons was carried out with the aid of compressors installed on the block. This operation was completed in four hours (six hours by the plan) and, at high water, made it possible to raise the block, which then had a draft of 5.7-5.8 meters (clearance 6.36 meters), which was 15 centimeters more than called for by the project. The pontoons were then flooded and the block once again settled onto the temporary bed.

11-3. Excavation of the Gap

Excavation of the gap was complicated by the greater (up to 40%) content of boulders in the soil than had been estimated from the geological surveys carried out by the Leningrad Planning, Surveying and Scientific Research Institute. The decision of the builders to construct the embankment using soil from the excavation of the construction area (during which fine fractions were washed away while the larger ones remained) resulted in a significant increase in the volume of excavation and complicated the operation of the dredge, which was not adapted for this type of soil. As a consequence of this the gate was damaged in the first few days and the dredge could discharge soil only to the right side. After going through the soft ground the output of the dredge dropped from 4,000 m³ per day to 300 m³ per day. Due to the absence of severe storms, however, it was possible to operate in the bay when the speed of water flow was quite high (up to 2 meters per second), which made it possible to avoid dredge idle time, as estimated and called for in the plan, and to complete the work

during the scheduled period of one month. On 8 August the captain of the dredge gave notice that the excavation was complete (Fig. 11-3). However, repeated and careful sounding measurements, followed by a stiff sweep, showed that it was necessary to carry out additional excavating operations, and even these did not permit complete clearance of the gap to the design width and depth (4.3 meters from the theoretical null depth).

In order to excavate large individual boulders it was necessary to use a floating crane and a tractor, which removed the boulders after their detection by sounding measurements and examination by divers. This work required great effort and organized interaction by various organizations and, although there was a tangible result, it did not fully solve the problem.

/234/

In order not to miss the period of syzygial tides, it was found necessary to decrease the required depth of excavation from 4.3 to 3.6 meters which, at a calculated tidal level of 3.9 meters, resulted in a decrease of the time available for the block to pass through the gap: 90 minutes instead of 120 minutes.

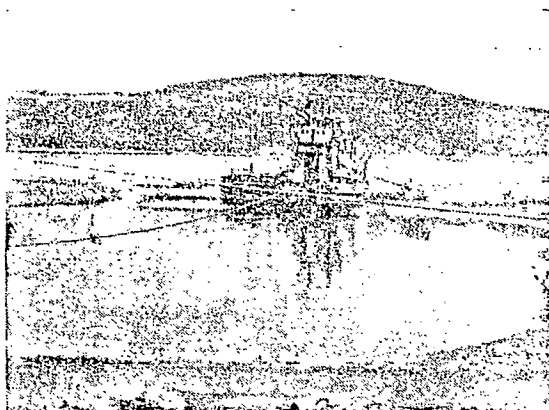


Рис. 11-3. Разработка прорези в завершающей стадии. 9/VIII 1968 г. Фото А. С. Фирфарова.

Figure 11-3. Final stages of the excavation of the gap. 9 August 1968. Photographed by A.S. Firfarov.

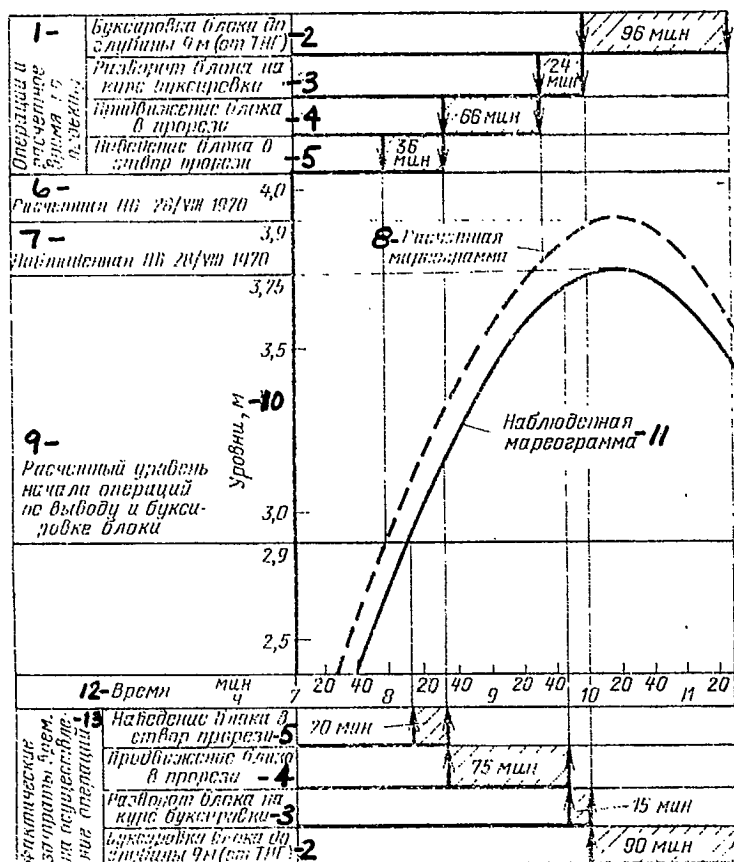


Рис. 11-4. Оперативный план-график вывода блока через прогону и сопоставление его с фактическим ходом операции.

Figure 11-4. Operational plan for bringing the block through the gap and comparison of it with the actual operation.

1 - operations and times allotted for them by the project; 2 - towage of the block to the 9 meter depth (from the theoretical null depth); 3 - turning of the block in the direction to be towed; 4 - movement of the block through the gap; 5 - aligning the block with the gap; 6 - calculated high water, 28 August 1970; 7 - observed high water, 28 August 1970; 8 - estimated tidal curve; 9 - estimated level at the start of removal and towage of the block; 10 - levels, meters; 11 - observed tidal curve; 12 - time; 13 - actual time required for carrying out the operations.

In view of the fact that at the start of removal of the block, when the level of the tide was 2.9 meters ($2.9 + 3.6 = 6.5$ meters, which leaves a margin of approximately 20 centimeters for the overall draft clearance of the block), and there was little probability of a bilge pad striking a boulder located at this depth, it was observed that diagonal

movement of the block toward the gap (instead of L-shaped movement, as stipulated in the calculations) would save time and, should a bilge pad graze a boulder, a waiting period for the tide to rise would come into effect.

/235/

After 28 August there remained only two days of syzygial tides of sufficient level to permit passage over the shallow section in the throat of Kislaya Guba and for this reason it was decided to move the block out of the construction area prior to the morning high water of 28 August 1968 at 1030 hours. Consultation on the eve of departure with the State Institute of Oceanography, the Leningrad Branch of the State Institute of Oceanography and the Administration of the Hydrometeorological Service, made it possible to correctly forecast a high water level of 3.75 meters (instead of the calculated 3.9 meters) and to recompute the plan for this amplitude without significantly altering the course of the operation. All preparatory work having to do with removal of the block had been completed by this time (Fig. 11-4). After soundings were conducted and all individual boulders which could impede the removal of the block were detected¹, these boulders were removed by tractor and floating crane using lines attached by divers. A final stiff sweep of the area was then carried out. At 0300 hours the decision was made to begin the towage operation with the morning tide of 28 August 1968 (Fig. 11-5).

¹These operations were carried out by crew No. 44 of the Leningrad Branch of the All-Union Planning, Surveying and Scientific Research Institute under the direction of M.L. Monosov, Engineer.

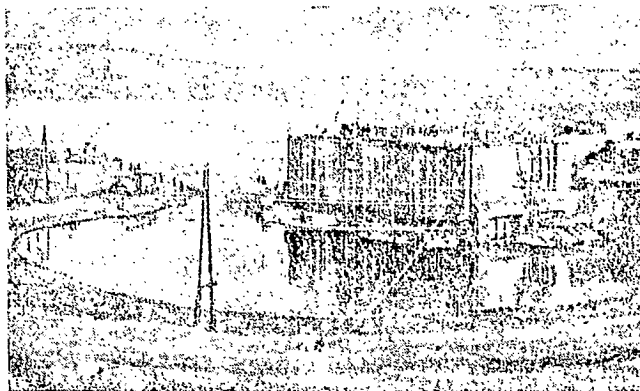


Рис. 11-5. Здание ТЭС перед выводом из прорези. 25/VIII 1968 г.
Фото В. Л. Шпаера.

Figure 11-5. The TPP structure prior to removal through the gap. 25 August 1968. Photographed by V.L. Shpaer.

11-4. The Removal, Towage, Installation at the Site, Submersion and
Ballasting of the Block

Removal of the Block. At 1700 hours on 28 August two diesel-
electric 2,000 horsepower tugboats and accompanying ships (rescue, fire
fighting launch and diving boat) anchored in the roadstead of the dock.
In contrast to the plan, in order to assure high manoeuvrability of the
block and in order to speed up its removal, a 350 horsepower canting tug,
equipped with a vertical-axis rotor, entered the dock. Another reserve
tug of this type anchored in the roadstead (it later accompanied the con-
voy until deep water was reached).

/236/

At 0748 hours, after the pontoons were blown out, which operation
began at 0330 hours, the block began to float at a level of 2.54 meters
above the theoretical null depth. At 0800 hours, at a tidal level of +2.70
meters (20 centimeters below the calculated level) and with a maximum
draft of 6.15 meters, the block slowly began to move towards the gap.

Manoeuvring with the aid of the winches and the canting tug made it possible to move the block to the gap by 0827 hours, i.e. 39 minutes later. By this time the water level was 3.10 meters above the theoretical null depth. The distance which the block had to cover to the gap was 136 meters. Eighteen minutes later (at 0845 hours), as the block was entering the gap at a level of 3.45 meters, the block ran aground and the left winch went out of order. Its rated load was exceeded at that moment, The progress of the block was immediately halted. The cable from the malfunctioning winch was detached from the anchor after several minutes and attached to the winch of a 40-ton tractor which was on reserve in the construction area. The other end of the cable was attached to a bollard on the left side of the block. Progress of the block was resumed and at 0930 hours it was fully within the gap.

At 0936 hours the prow hawsers were released from the buoys in the roadstead and at 0946 hours, i.e. one hour and 58 minutes after the start of movement, at a tidal level of 3.65 meters, the block came out of the gap and the shore hawsers were immediately released. The block began to be pulled by the tugboat's cable winch. From this time until 1000 hours, simultaneously with the forward movement of the block as the result of the tugboat winch, the stern canting tug turned the block around until it was in position for its trip (Fig. 11-6).

This manoeuvre was carried out over a depression with depths to 6.5 meters from the theoretical null depth, where it was initially proposed to hold the block (in case there should not be enough time for it to traverse the shallows during high water). But since removal of the block was completed before the advent of high water and there was enough extra time to traverse the shallows with the tidal wave (although the high water level was 15 centimeters below the previously calculated level), it



Рис. 11-6. Разворот блока в исходное положение. 10 ч 28/VIII 1968 г. Фото А. С. Фирфорова.

Figure 11-6. Turning the block to its starting position. 1000 hours, 28 August 1968. Photographed by A.S. Fifarov.

was immediately decided to keep moving and traverse the 4.8-kilometer long shallows.

Towage of the block. Since towage through this area was carried out at decreasing tidal speeds, the shallows were traversed at sufficient depths prior to high water. This was achieved with the aid of the lead 2,000 horsepower tugboat. The speed of the convoy was 2 knots, which was below the speed obtained for this section (2.8 knots) during model studies carried out by the Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute.

At this time, with a towing cable length of 55 meters, yawing of the block of up to 10° was observed in relation to the fore-and-aft line. After the shallows were traversed, at 1200 hours, a second 2,000 horsepower tugboat joined the convoy, as the result of which the speed increased to 3.5 knots. In order to decrease yaw the towing cable was lengthened to 120 meters.

The weather was favorable during the crossing of Kola Bay (sunny, southerly winds of force 1-3). After leaving Kola Bay (2045 hours) a slight swell appeared, the block began to yaw, the heel attained 5°, trim 4°, and waves up to 1.5 meters. The speed was then decreased to 3 knots and the towing cable was lengthened to 250 meters. At 0045 hours the lead tug left the convoy.

/238/

At 0015 hours the wind strengthened to a force of 4, having changed to a southeasterly direction, at which time the speed was decreased to 1.7 knots and the towing cable was taken up to 95 meters. At 0217 hours on 29 August the convoy left the open sea, entered Ura Guba and passed through the narrows between the islands of Shalim and Shurinov. The force of the wind decreased to 3, the speed of the convoy was increased to 2.5 knots. As the Kislaya Guba roadstead was approached at 0420 hours, the towing cable was taken up to 45 meters and the speed was decreased to 1 knot. At 0447 hours the convoy reached the roadstead. Moorage of the block to the buoys of the Kislaya Guba roadstead was begun. Large blocks (50 and 10 tons) were previously installed by a work boat with a prow crane. At 0525 hours the tugboat took up the cable, moved away from the block and moored to a buoy in the roadstead. At this time the block had a heel of 7 centimeters, a trim by the stern of 17 centimeters, and its draft was close to the rated draft.

At 0601 hours the block was made fast to the roadstead buoys with the aid of two capron cables. At 0645 hours the capron cables were replaced with steel cables from the stern winches while the cables from the two prow winches were attached to shore ring bolts installed in the throat of Kislaya Guba (Fig. 11-7).

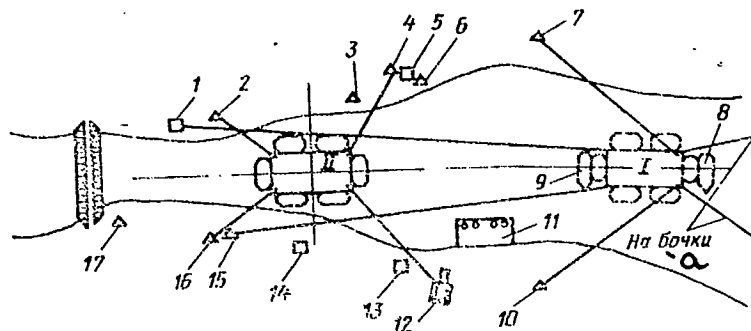


Рис. 11-7. Схема ввода блока в створ.

I - блок в исходном для ввода положении; II - блок в створе; 1, 5, 13, 14 - массивы с петлей для удержания блока в створе; 3, 6, 7, 10, 15, 17 - рымы в скале для удержания блока в створе; 2, 4, 16 - рымы в скале для удержания и погружения блока в створе; 8 - буксир-кантоувальщик мощностью 350 л. с.; 9 - катер грузопассажирский мощностью 150 л. с.; 11 - постоянный причал с четырьмя причальными тумбами; 12 - экскаватор Э-1252 для удержания и погружения блока в створе.

Figure 11-7. Schematic diagram of the entry of the block.

I - block in the initial position for entry; II - block in the TPP site;

1,5,13,14 - heavy blocks with rings for holding the TPP block in position; 3,6,7,10,15,17 - ring bolts for holding the TPP block in position; 2,4,16 - ring bolts in the cliff for holding and submerging the TPP block; 8 - 350 horsepower canting tugboat; 9 - 150 horsepower passenger and freight launch; 11 - permanent berth with four bollards; 12 - E-1252 excavator for holding and submerging the TPP block at the site; a - to buoys.

Installation at the TPP site. After a five-minute pause, at 0656 hours, the block began to be moved from the roadstead anchorage to its resting place and at 0740 hours it was 170 meters from its destination. When the block was 130 meters away the left prow winch went out of order as the result of loose mounting of the reduction gear (the cause was the same as in the first instance with the left stern winch). After repair of the winch, which lasted until 0900 hours, the prow cables were taken in to the starting position for moving the block to its resting site. The cables were transferred from the boat to the launch "Gidrostroitel", which completed attachment of the cables on the west bank at 1053 hours, and on the east bank at 1115 hours. This loss of time (2 hours) for winding in the cables is explained by the fact that the low speed of the winch was

not taken into account and a cable of the required length was not unwound in advance.

/239/

From 1115 hours on, the block was moving toward its destination as the water level was dropping, since high water had occurred at 1045 hours. At 1145 hours the progress of the block was again halted because the clutch had snapped, after which the winch could only play out the cable. At this time the end face of the block was at a distance of 50 meters from the normalizing bar, which has a depth of 3.45 meters (from the theoretical null depth), while during ebb the level had dropped to the 3.01 meter mark. The actual depth at the normalizing bar was $3.01 + 3.45 = 6.46$ meters. Movement toward the normalizing bar with the aid of the winches, taking stops into account, was to have taken 1.5 hours (96 minutes), during which time, as the result of ebb, the sea level would have dropped to 2.25 meters above the theoretical null depth and the depth would have been $2.25 + 3.45 = 5.70$ meters, which would have ruled out the passage of the block which has a draft of 6.30 meters. For this reason, at 1200 hours the block was returned to the starting position and at 1238 hours it was secured with hawsers (Fig. 11-8) in order to bring it in at 2317 hours of the same day during the evening high water, which was supposed to have had a height of 3.16 meters. At 2046 hours, at the peak of the flood, the block betan to be moved toward its resting place (Fig. 11-9). V.F. Kuligin, the commander of the towage operation, was in charge. He placed a tugboat at the stern to keep the block from yawing, while the launch "Gidrostroitel" was moored to the prow pontoon. As the block moved toward the normalizing bar the level of the tide was radioed at 10 minute intervals from the shore to the block command post. These levels were converted to the margin of water depth beneath the bilge pads on either

/240/

side of the block. By using these data and by skillfully directing the operation of the stern and prow tugs, the commander of the towage operation brought the block across the shallow section while the margin was minimal (Fig. 11-9,b), and with minimal loss of time spent in waiting for the necessary level of the tide. The operation of bringing in the block was carried out on the basis of a zero clearance beneath its bilge pads. This made it possible to have a time margin for unforeseen stops or for decreasing the margin necessitated by non-periodic components of the tide (off-and-on water phenomena, atmospheric factor). In our case the non-periodic components could have resulted in a 0.30 meter decrease in the level. In actuality, off-water was not observed.

It was considered that it would be safe for the block to touch the bottom as it was being brought in because the area of limiting depths was made up of soft (loamy) soil. When the block touched bottom it was immediately stopped and then, as the tide came in, it was moved closer to its destination.

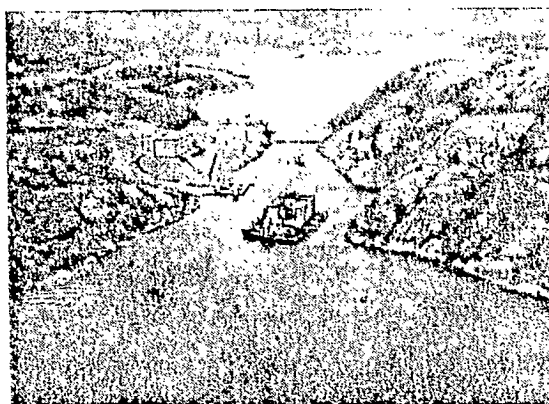


Рис. 11-8. Первая попытка ввода блока в створ.
12 ч 29/VIII 1968 г. Фото В. Л. Шпаера.

Figure 11-8. First attempt to bring in the block. 1200 hours, 29 August 1968. Photographed by V.L. Shpaer.

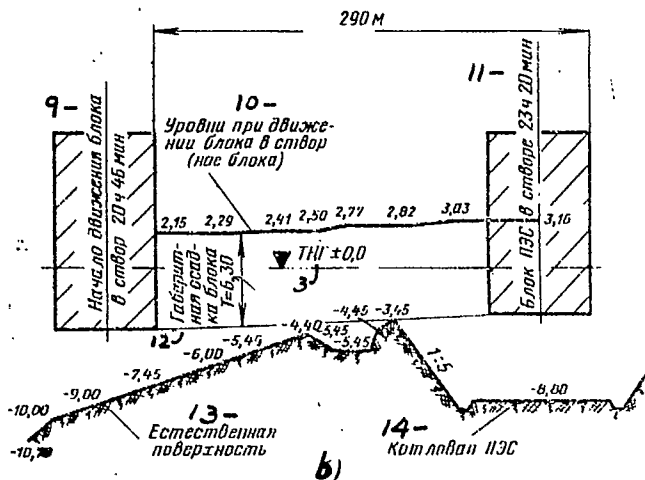
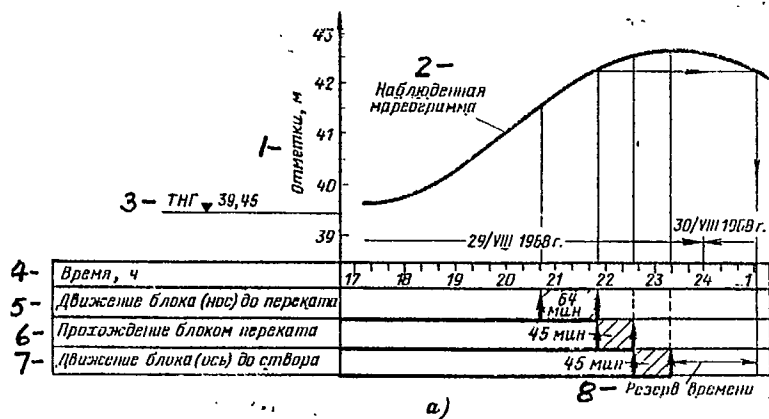


Рис. 11-9. Завод блока в створ ПЭС.

а — оперативный план-график; б — проход блоком нормирующего переката.

Figure 11-9. The operation of bringing the block into the TPP site.

а — operational graph; б — passage of the block over the normalizing bar.

1 — level marks, meters; 2 — observed tidal curve; 3 — theoretical null depth; 4 — time, hours; 5 — movement of block (prow) to bar; 6 — crossing the bar; 7 — movement of the block (center line) to its resting place; 8 — time margin; 9 — start of movement of block to its resting place at 2046 hours; 10 — water levels during progress of the block to its destination (prow of block); 11 — TPP block at its destination, 2320 hours; 12 — overall draft of block, $T = 6.30$; 13 — natural surface; 14 — TPP foundation area.

On 29 August, at 2320 hours, the block was made fast at its destination (Fig. 11-10) using the block on the shore 1, the ring bolts 4, 15 and the bollard (Fig. 11-7). The operation lasted two hours and 34 minutes, which gives an average speed, taking stops into account, of 0.95 meters per minute (planned speed 0.9 meters per minute). The maximum speed reached

3 meters per minute, but it was lost as a result of the need to wait for the tide to rise at the bar and the time needed to rewind the cables.

During the night of 30 August the north wind, which went up to force 5, tore out ring bolt 15 (Fig. 11-7) because it was insufficiently deeply embedded in the cliff. The block began to move toward the west bank and the cable was moved to spare ring bolt 17 (Fig. 11-7). As the strength of the north wind increased above a force of 5 during the evening of the same day, concrete block 5 began to move. The hawser was moved to a spare ring bolt but the unreliability of the ring bolts made it necessary to call out a rescue tugboat, which arrived 27 minutes later and, while anchored, held the block in place by means of a capron line in conjunction with the other cables. Subsequently, on 31 August, three secure ring bolts 2, 4, 16 were drilled into the cliffs (Fig. 11-7), in order to reliably hold the block in place during its submersion.

/242/

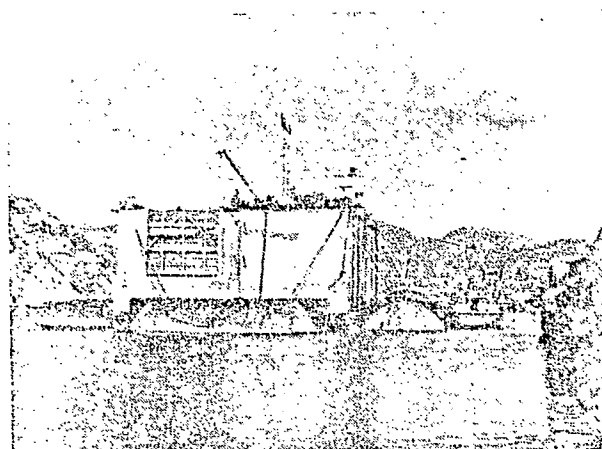


Рис. 11-10. Блок в створе ПЭС. 30/VIII 1968 г.
Фото А. С. Фирфарова.

Figure 11-10. Block at the TPP site. 30 August 1968. Photographed by A.S. Firfarov.

On 31 August, at 2020 hours, while air was being bled out of the pontoons, the pontoon on the northwest corner of the block unexpectedly snagged on the cliff. This occurred as the result of uneven flooding of the pontoons, which resulted in their being jerked about. The block became propped up on this pontoon and immediately acquired a heel of 4° . The incoming tide soon rectified the situation but, in order to prevent the block from getting caught on the pontoons during ebb, work was begun on cutting the eye splices of the supporting slings. This was completed by 0100 hours on 1 September. During the same night, at 0230 hours, all of the side pontoon cables were removed and pulled onto the shore with the aid of a bulldozer and a 10-ton floating crane. The bilge pads were removed during the subsequent 24 hour period (ending at 1200 hours on 2 September). After the underwater attachments were loosened by divers, the bilge pads were placed in slings and pulled onto the shore with the aid of a tractor or with the aid of the capstan of the floating crane. Thus, at 1200 hours of 2 September, the block was ready to be submerged.

/243/

Submergence of the block. On 2 September, at 1443 hours, after the block was securely fastened to three ring bolts located at a 45° angle to the axis of the block, and a fourth line was made fast beneath the treads of an excavator (holding block 13 was poorly anchored), and after the block was precisely positioned at the stie, it was begun to be lowered onto its foundation.

In accordance with the plan, before lowering the block onto its foundation, at the start of the ebb tide, the draft of the block was increased by means of ballast to 1.55 meters, which made it possible to settle the block 1.5 hours before low water. The assurance of correct settlement onto the bed was achieved by timely alignment of the block and

by selection of cable slack as the water level dropped. There was constant instrumental and visual control of the position of the block both from shore and from the block itself. An important role in the accurate settlement of the block was played by the proper arrangement of the shore anchors in the vertical (at approximately the same elevation) and horizontal (angles with the axis were 45°) positions, as well as the choice of slow winches, which made possible smooth positioning of the block at the site.

After instrumental assessment of the horizontal and vertical settlement of the block, additional water ballast was taken on into the suction pipes and the spillway section (Fig. 11-11). The maximum horizontal deviation of the block was 0.19 meters, which is significantly less than permissible in this instance. However, as was established by the submergence of the block at the construction dock and in Kislaya Guba, it is relatively easy to ensure a horizontal precision of approximately 0.20 meters. This shows that, under conditions for industrial TPP's, precise positioning of the blocks at their proper locations is a problem which has been solved.

The vertical position of the block after settlement deviated from the plan at the four corners by -32, +20, +1 and -13 millimeters.

After stabilization of the vertical position of the block the mean overall settlement was 25 millimeters, with a difference of only a few millimeters at the corners, and only 1 millimeter per year during the course of 1970-1971, which confirmed the correctness of the design solution and the high quality of the construction of the bed. /244/

The general management and execution of the operation, and the involvement of many organizations in its development and execution, were handled by the Chief Project and Construction Engineer, who was also designated Chief Engineer of the towage operation.

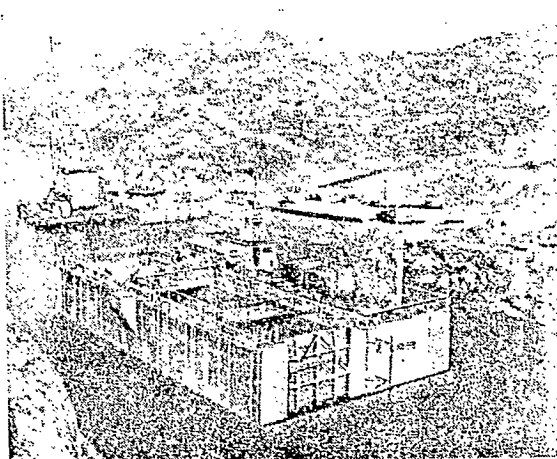


Рис. 11-11. Блок ПЭС после погружения на основание. 3/IX 1968 г. Фото С. Л. Гельфера.

Figure 11-11. The TPP block after settlement onto its foundation. 3 September 1968. Photographed by S.L. Gel'fer.

In addition to the design and research institutes listed previously, the following took part in carrying out the operation: a group of marine experts under the direction of V.F. Kuligin (commander of the towage operation) with the participation of V.N. Bryushinov, M.I. Burkov, V.G. Bakharev, V.I. Ivashchenko, I.I. Firsov, A.F. Sterneichuk and others, who directed the block, shore, hawser, pontoon, and engine crews and carried out the installation, blow-out, and removal of the pontoons, the operation of the winches, and the running lines by boats and making them fast to anchors and buoys. All ships and diving devices taking part in the operation were under the orders of the commander of the towage operation. The following services took part in the execution of the operation:

1. The hydrographic service, which handled the conditions in the channel in the shallow sections along the course, the sounding of the course, and the stiff sweeping of the gap.

2. The hydrometric and hydrological services (Moscow University Hydrometeorological Service, F.S. Terziev, Director; Leningrad State Institute of Oceanography, Group No. 44, Leningrad Planning, Surveying and Scientific Research Institute), which provided continuous information about the weather and water levels, and carried out accurate sounding of the gap and the entrance channel as well as hydrological control during the removal and installation of the block.

/245/

3. Power, mechanical and rescue-construction services, as well as housekeeping services (Northern Administration for Hydraulic Engineering Construction).

4. The services of scientific and research and field observations over the conduct and stressed condition of the construction (Scientific Research Department of the All-Union Planning, Surveying and Scientific Research Institute, Main Administration for Rural Power Systems Design, All-Union Scientific Research Institute of Hydraulic Engineering, and the Main Distributing Point).

5. Author's supervision of the towage operation (Engineers N.M. Nekhoroshev and S.L. Gel'fer). N.M. Nekhoroshev together with A.I. Burkov drew up the plan for removal of the block from its construction site and revised the execution of the towage plan; S.L. Gel'fer personally directed ballasting of the block and conducted all operational computations for this procedure.

6. Information Services (the press, radio, television, motion pictures).

The harmonious and efficient work of all of these organizations and participants in the operation ensured its successful execution.

The main conclusion which may be drawn from the towage operation is that the possibility of constructing tidal and low-head river hydroelectric power plants by the floating method and the correctness of the developed project have been proven, even though the rates used in calculations and obtained from model studies were found to be too high, while the power of the tugboats called for in the plan was too low. Despite this, however, the cost of the operation (86,000 rubles) was lower than estimated (115,000 rubles). This is explained by the favorable meteorological conditions under which it was not necessary to cover the cost of idle time of the ships. It is important to take into account, however, that this sum, just as the estimated sum, is but a small part of the total estimate and in the case of large-scale construction will not be decisive in comparison with the obvious advantages made possible by the floating method.

Ballasting. In accordance with the plan, replacement of the water ballast with soil was to have begun immediately after settlement of the block onto its foundation. In order to do this it was necessary to dump 900 m^3 of a sand and gravel mixture from the Ura Guba quarry to a height of 6.5 meters from the bottom plate. Above this level, from the zone of variable (tidal) levels to the top of the structure (46 meter mark) ballast composed of hydrophobic soil was to have been dumped. The urgent need to execute the first part of this task was dictated by the fact that after settlement of the block, which was carried out on 2 September during the period of quadratures in accordance with the project, the tides began to grow, destined to reach their maximum height on 9 September (syzygial amplitude 3.6 meters). Under those circumstances the stability of the block against floating up while ballasted only with water was insufficient (stability coefficient equal to 0.8) and 900 m^3 of soil had to be packed

into the spacings of the block in six days. The difficulty of the problem stemmed from the fact that in order to deliver this soil it had to be obtained from the Ura-Guba quarry, loaded into dump trucks by means of an excavator with a 0.6 m^3 capacity bucket, transported 6 kilometers to the wharf, loaded onto a barge by means of a 10-ton truck crane, towed along the bay, unloaded from the barge with the aid of the floating crane's grab bucket, and finally dumped into the block spacings.

All of this had to be accomplished quickly during the course of a few days because otherwise the block would float up and it would probably not be possible to replace it as precisely and evenly as called for in the project. Also of primary importance was the fact that September is the month of autumn storms which could have closed the water route to Kislaya Guba for the delivery of the ballast soil.

This is why this work had to be organized as a military operation, giving it control of all construction resources and powers. Since the bottleneck in this cycle was the transport of the soil along the bay, in order to tow two 100-ton scows the main passenger-tug-launch "Gidrostroitel" with a 150 horsepower engine and equipped with radar for sailing through fog, was used. Later, during the ballasting period, an RBT-70 harbor tug with two 150 horsepower engines, obtained from Murmansk, was used.

The turnaround cycle for the tugboat took an average 10-12 hours: two hours - towage in one direction, 2-3 hours - unloading, 2 hours - return, and 3-4 hours - unloading (depending on delivery of the soil). For this reason, depending on the delivery of the soil, one tugboat could make 1-2 trips per day, i.e. deliver $50-100 \text{ m}^3$ of soil, while two tugboats could not make more than 3 trips per day. Consequently, in order to deliver the minimum requirement of 900 m^3 prior to 9 September (in six days) it was

necessary to establish a rigid schedule of soil delivery to the wharf and of the movement of the tugboats. In order to accomplish this, in the area of the quarry on the outskirts of the settlement of Ura-Guba, a special motorized column was deployed consisting of: 10 dump trucks, an excavator, a bulldozer, a grader, a truck crane, and a portable generator. The personnel were billeted in trailers and were under the command of the column commander. He was responsible for round-the-clock loading, delivery to the wharf and loading of the soil onto the barges regardless of weather conditions. During periods of bad road conditions a bulldozer was available for pulling out stuck trucks.

/247/

The next bottleneck, reloading onto the scow, was overcome by a simple technique. The soil from the truck was dumped into a trough with four hooks onto which the sling from the truck crane was attached. After the trough was lowered onto the pontoon the sling-man would unhook one side (two cables) and the trough would empty as it was lifted off the pontoon. The scows could only be towed at a wind force of up to 3-4 from the northwest. However, the captain had to base his decision on whether or not to halt towage not on information about the actual weather but rather on a forecast for the entire Murmansk Coast. Since this information differed significantly from the actual weather in the bay along the course of towage, by special arrangement with the Murmansk Directorate of the Hydro-meteorological Service, there was a weather forecaster right on the ship who determined the force and direction of the wind. This made it possible to decrease idle time due to weather to a minimum. Barges were delivered almost continuously to the block where the floating crane unloaded the ballast directly into the spacings (Fig. 11-12). The organization of this flow of soil made it possible in the shortest possible time, in spite of

severe weather conditions, to counteract the lifting force of the syzygial tide by means of the necessary ballast weight, which ensured the block's stability against floating up.

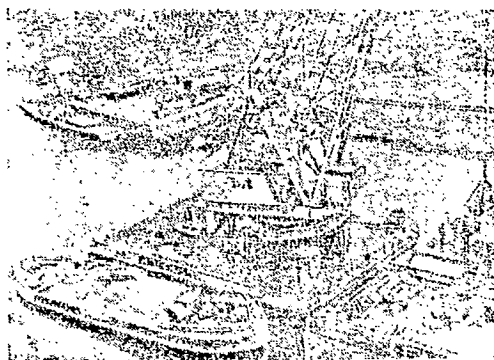


Рис. 11-12. Разгрузка плавучим краном
грунтового балласта и подача его в яма-
ны.

Figure 11-12. Unloading of the soil ballast and dumping of it into the spacings by means of the floating crane.

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/247/

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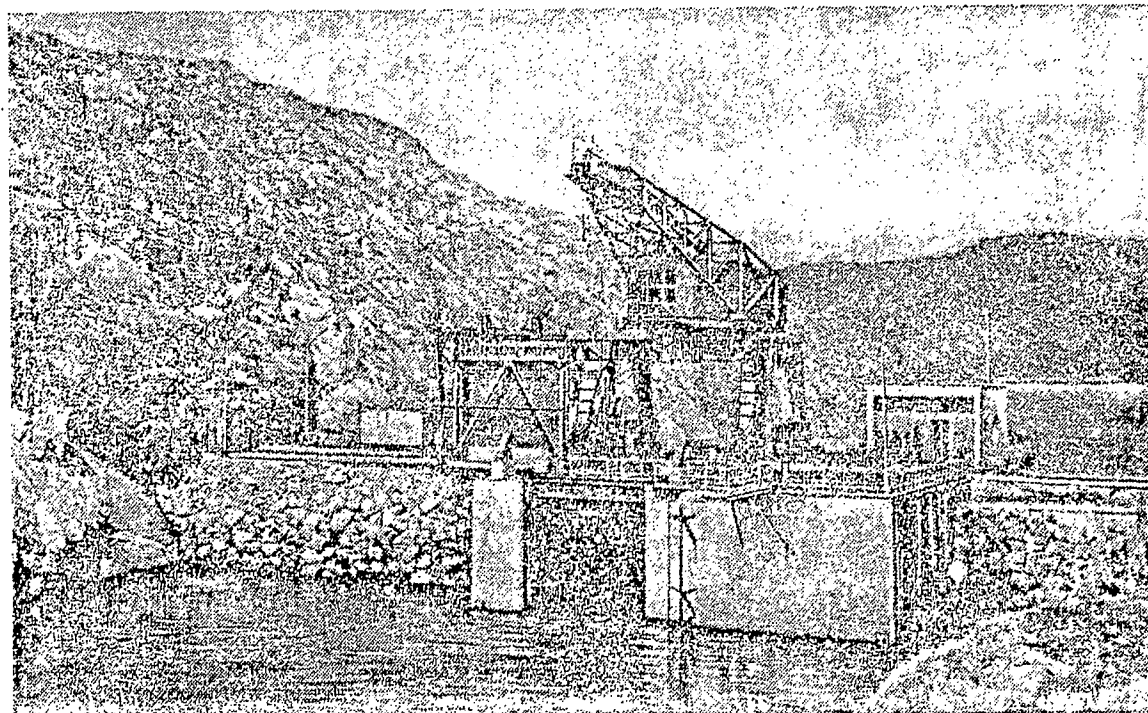


Рис. 12-1. Кислогубская ПЭС. 1970 г. Фото Л. Б. Бернштейна.
Обратный турбинный режим, работает также водослив.

Figure 12-1. The Kislaya Guba TPP. 1970. Photographed by L.B. Bernshtein. Reverse turbine regime, spillway also in operation.

Chapter 12

THE HYDROTECHNICAL SIGNIFICANCE OF THE EXPERIMENT OF CREATING THE KISLAYA GUBA TPP

- The Kislaya Guba TPP (Fig. 12-1) has been operating for three years /249/ as part of the Kola Regional Administration of Power System Management producing, during non-experimental periods, 100,000 kilowatt hours per month. This small output, of course, is not significant in the electrical balance of the system, but its importance lies in the fact that these are the first kilowatt hours obtained in Soviet energy grids from the sea.

The Kislaya Guba TPP, as any experimental installation, cannot serve as a measure of the efficiency or specific indices of tidal energy. Its small capacity cannot justify the relatively large outlays which were required to solve problems extending far beyond the confines of its own significance.

It should also be remembered that tidal energy, which originates in the expanses of the World Ocean, as indicated by research [3], cannot be "locked" in a small installation intended to provide energy for a small local consumer. The damming of large areas of the sea cannot be economically justified if these dams are outfitted with low-capacity equipment. On the other hand, the effect from a tidal electric power plant obtained during its peak output capacity can only be realized when it operates jointly with powerful steam (excluding nuclear) electric power plants, i.e. in an energy grid encompassing large economic regions on the national or even continental scale. In the research indicated above [3] it was shown what a significant effect could be obtained by including super-powerful TPP's within the unified energy grids of Europe and America or in the integrated power system of the European part of the USSR.

The present state of power engineering in the European part of the USSR confirms the validity of these hypotheses. It is known that the surge of the evening peak of the unification of the power grids of the USSR is of the order to 30 million kilowatts. Since the capacity of river hydroelectric power plants is insufficient to cover this peak, a significant portion of it has to be covered by block steam power plants. It is evident even to non-specialists that such operation of super-powerful steam electric power plants for 3-5 hours per day is distinctly inefficient and one can clearly see the striking effect which could be obtained by including tidal power plants in an integrated power system which, by operating daily for 3-5 hours during the peak and by absorbing the idle capacity of steam power plants during the night, could significantly lower the cost of the energy grid. /250/

It is evident from the above that the significance of the Kislaya Guba installation is determined by the fact that in the field, on the scale of an actual structure whose dimensions are comparable with those of an industrial TPP, it has proven the possibility of building a TPP by the floating method. And this method alone already ensures tremendous savings in the cost of a TPP. The experience of building the Rance TPP, which is industrially significant (240 thousand kilowatts), has shown that 30% of the cost is taken up by the construction of the dam. The Rance TPP, however, was built under favorable climatic conditions in a populated region. Inasmuch as the sites of possible TPP construction in the USSR are located in unpopulated regions of severe climate, a decrease in the cost combined with removal of the need to construct cofferdams will be significantly more appreciable. The experience of building the Kislaya Guba TPP shows that work carried out under such conditions at a TPP site will be 2-3 times more expensive than under the favorable conditions of an industrial maritime center.

At the same time it ought to be taken into account that under these conditions of experimental construction, the small size of the building operations, and thus the impossibility of utilizing large scale and complete mechanization, resulted in inefficient and extremely expensive work. We will cite examples presented in earlier chapters during the description of carrying out the work of building the Kislaya Guba TPP.

The underwater excavation, as a result of the primitive nature of the work of loosening the bottom with the aid of concentrated charges, came to double that of present-day costs. This also applies to the work of consolidating the bed which, together with the cost of the excavation, cost one million rubles of the 6.25 million ruble cost for the entire construction.

The unit cost for concrete established at the start of construction set the cost of the concrete at 200 rubles per m^3 . The cost of the concrete as determined at the end of construction from actual outlays was 122 rubles per m^3 . Understandably, in the construction of a large TPP this cost could be significantly lowered by mechanizing all concreting operations and by using precast members. It should be noted that such construction could be made highly efficient by employing the completely pre-cast variant, which was impossible to achieve at earlier sites (Saratov, Kiev, Kanev, and Plyavinyas Hydroelectric Power Plants [45a]). This is explained by the fact that, with this method, construction is based on the use of thin-walled members which make it possible to replace the main bulk of the concrete with cheap ballast (sand).

/251/

The mechanical and hydraulic power equipment, each element of which is in effect unique and manufactured as a single sample, also cost several times more than would serial production for industrial installations. The equipment cost 580 thousand rubles and its installation 160 thousand rubles.

A total of 740 thousand rubles. 359 thousand dollars was paid to the manufacturer of the imported 400 kilowatt capacity tidal hydraulic turbine unit, weighing 100 tons, while the installation of the unit and the rest of the equipment cost another 90 thousand rubles. This excessively high cost is determined by the unique nature of the machine and its single-unit construction coupled with the low capacity of the unit due to the small heads and tidal amplitudes under the conditions at Kislaya Guba in comparison with potential construction sites for industrial TPPs (in the Gulf of Mezen, for example, the amplitude of the tide attains 10 meters in comparison with 4 meters at Kislaya Guba). For this reason, for example, under the conditions at the Mezen site, for the same dimensions and weight, the Kislaya Guba TPP could produce 3,200 kilowatts at a design head of 5 meters, which would make it possible to obtain a metal capacity of 25 kilograms per kilowatt at a cost of 1 thousand rubles per ton and, using serial production, this would result in 32 rubles per 1 kilowatt as opposed to 900 rubles per kilowatt in the case of Kislaya Guba, i.e. the unit cost of the turbine unit per 1 kilowatt of capacity under the conditions of an industrial TPP would be 28 times less than at the Kislaya Guba installation.

Another point to be considered here is the possibility of significantly lowering the unit cost by increasing the single-unit capacity of the turbines to 15 thousand kilowatts while increasing their diameter to 7.5 meters (this diameter for a bulb-type turbine unit has been realized in the field in the experimental turbine of the Saratov Hydroelectric Power Plant). In addition, the latest research on the design of TPP's in Canada and England [56] has shown the possibility of using tubular turbines or, in the case of a double-basin TPP scheme or single-basin unidirectional operation, simpler machines with a propeller runner wheel and an external generator on an inclined axis, manufactured at the present time by the American firm Allis Chalmers.

The high hydrothermal insulation cost of 125 thousand rubles (88.3 rubles per m^3) is also explained by the experimental nature of manufacture not only of the insulation but also of the raw materials, which took up 80% of the cost.

The construction dock which, in this instance, cost 1.7 million rubles (25% of the total cost of the TPP), should not be fully included in the general estimate for the TPP at all since the dock will, in the future, after completion of construction of the TPP, be employed for the needs of shipping and shipbuilding (in this case the construction dock was transferred to the fishing industry and 1.3 million rubles of the cost of its construction have been taken off the construction cost of the TPP).

The 1.5 million ruble cost of the scientific research, and planning and surveying work cannot be relegated to the costs for the installed capacity since these costs were necessitated by both the study of the problem as a whole and by the independent significance of this work toward the solution of a series of urgent problems of hydraulic engineering construction in the north.

Thus, if we exclude from the cost of construction of the Kislaya Guba TPP the costs of the scientific research, and planning and surveying work and the cost of the construction dock, which was removed from the balance sheet, then the cost (including the second turbine unit) for the design capacity of 800 kilowatts (for which the TPP structure was built and all equipment was designed) will be 7,360 rubles per kilowatt. As a first approximation it may be reckoned that if the Kislaya Guba turbine unit were installed under the conditions at Mezen this cost would be lowered, i.e. 5.9 million rubles: 6.4 thousand kilowatts = 920 rubles per kilowatt. The concrete possibility of a significant decrease in the cost of a TPP structure built on an industrial scale as a result of the difference

in the cost of basic operations (the cost of underwater excavation and concrete halved) shows that the total size of specific capital investments for the construction of powerful TPP's may also be decreased in comparison with the data presented for the Kislaya Guba TPP by at least twofold and will come to 500 rubles per kilowatt, which is comparable to the cost of river hydroelectric power plants in this region.

Understandably, this figure cannot at this time be included in the economic substantiation for the construction of a powerful TPP. It is quite clear that in order to accomplish this it is necessary to carry out research, put together a project and work out its power engineering economic substantiation. Studies of the Gulf of Mezen for future TPP's, which were jointly carried out by the Main Administration of the Hydrometeorological Service of the Council of Ministers, USSR and the Institutes of Oceanology and Oceanography, gave a clear idea of the obstacles which must be overcome in order to build a Mezen TPP. These include ice, detrital deposits, stormy seas and low temperatures. The experience of building the Kislaya Guba TPP, however, inspires confidence and hope in the possibility of a successful solution to these problems not only along the White Sea coast but also in the more distant future along the Sea of Okhotsk. The now developing possibility of obtaining, for example, from the first Mezen TPP four million kilowatts of guaranteed peak capacity with an output of 7.5-10 billion kilowatt hours per year, as well as the prospect of consuming the free night-time capacity of superpowerful steam power plants to be built in the operating regime of the Mezen TPP, indicate the undoubtable expediency of designing this installation. In accordance with a directive from the Ministry of Power, USSR this task has been taken up by the team which created the Kislaya Guba TPP. Concurrently with this, at the Kislaya Guba

/253/

TPP there will be a continuation of the research necessary for further work on the industrial utilization of tidal power and for various subdivisions of hydraulic engineering construction.

Research will continue in this manner toward the solution of the following problems, work on which has already resulted in significant savings.

I. Creation of materials with very high frost resistance:

a) concrete, created and tested at the Kislaya Guba TPP, has high strength (800 kgf per cm²), impermeability and frost resistance (withstood 5,000 freezing-thawing cycles, equivalent to 6,200 standard cycles), which makes it possible to use it at this time for harbor structures in northern tidal seas and to completely replace short-lived wooden berths and metal structures. Further testing will make it possible to confirm the service life indicated by accelerated tests and to examine other concretes under these conditions (including sand concrete).

/254/

In order to use concrete under the more severe conditions of the White Sea coast (Mezen) it is necessary to test the concrete further for durability, frost resistance, strength and increased resistance to the abrasive action of ice;

b) epoxy from hydrothermal insulation underwent the same number of accelerated cycles and may safely be used under conditions similar to those at the TPP in order to decrease temperature drops and to smooth out their fluctuations. Further studies will also make it possible to test the data, obtained from accelerated cycles, about durability and to determine the degree of deterioration of the insulation with respect to its thermal insulation properties (change in thermal conductivity with time), to conduct a search for ways of increasing the strength of the insulation against mechanical attack as well as to eliminate technological causes resulting in increased thermal conductivity in the zone of variable levels;

c) the hydrophobic soil used at the Kislaya Guba TPP may be recommended for ballasting hollow structures in the zone of variable water levels as a reliable material which protects the structure from damage.

II. Protection of the structure and equipment from the corrosive action of water:

a) the cathodic protection used at the Kislaya Guba TPP has shown its effectiveness against electrochemical corrosion. The main aim of further research is to determine the effectiveness of this protection for the reinforcement of reinforced concrete (preservation of the bond with the reinforcement) and the establishment of optimal regimes in the presence of simultaneously operating cathodic protection with anti-fouling coatings;

b) the anti-fouling coatings used at the Kislaya Guba TPP are basically the same as, or slightly modified versions of, coatings used in shipbuilding. The continuation of research begun on new more durable coatings should produce a noticeable effect in shipping and shipbuilding also.

III. The power engineering and mechanical equipment:

/255/

a) the bulb-type turbine unit at the Kislaya Guba TPP served as the basis for the design and creation of such units at river hydroelectric power plants. The Soviet machine building industry not only mastered their construction and supplied 54 units to the Kiev, Kanev, Cherepovets, and Inguri TPP's, with a total capacity of 900 thousand kilowatts, which resulted in a significant economy in the cost of construction and had a significant power engineering effect, but also manufactured new improved types of these machines (the Cherepovets unit with water cooling of the rotor and stator, and the Saratov unit with the world's largest cantilevered wheel).

These achievements have made it possible for the USSR to change from an importer of bulb-type hydraulic turbines to an exporter of them and to successfully compete on the world market with French companies (the inventors of the bulb-type hydroturbine unit), and to obtain and deliver an order of such machines to Norway. There is no doubt that successful testing of the bulb-type unit now manufactured for the Saratov Hydroelectric Power Plant will make possible the wider use of these machines in the USSR.

Study of the Kislaya Guba TPP unit will make it possible to determine the optimal regimes for the utilization of tidal energy. In addition, work is being done on the creation of a variable speed hydraulic turbine which will make it possible to increase the efficiency of TPP's and low-head river electric power plants operating under relatively highly variable heads.

b) research on a new system of oil heating for gate guides will make it possible to increase the manoeuvrability of power plant gates under severe climatic conditions, which is of real significance for planned superpowerful hydroelectric power plants in Siberia, Kolyma, and Kamchatka.

IV. The new design of the Kislaya Guba TPP structure has shown the possibility of delivering it by floating it in, and of significantly decreasing (by 2-3 times) the expenditure of concrete. Additional full scale study and development of this system are needed to apply it at the Mezen TPP project and to construct low-head hydroelectric power plants by the floating (and usual - construction in a foundation pit) method. The experience of building the floating structures of the Kislaya Guba TPP will be especially significant in the construction of hydraulic power structures in connection with the forthcoming development of the Arctic Ocean shelf.

The special devices and methods of preparing the underwater foundation which were developed and used for installing the floating block have proven their effectiveness and reliability (settlement of the block proceeded smoothly, was found to be insignificant, and stabilized after a few months; the floating depth gauge, marker floats and leveling rod made it possible to quickly prepare an underwater bed of high quality) and may be recommended for wide use in underwater engineering operations. Further research and development in this field should provide full scale testing of the mechanized apparatus for laying the bed, and development of a mechanized method for underwater excavation of rocky and soft soils using floating drilling installations of the "Khazar" and "Apsheron" types.

It is evident from the above that construction of the Kislaya Guba experimental TPP had an important effect on the adoption by the national economy of a variety of solutions worked out at this TPP.

By decision of the Ministry of Power, USSR, the Kislay Guba TPP is an experimental base and research laboratory on the problem of utilizing tidal energy and hydraulic engineering construction in the north.

The significance of the Kislaya Guba TPP for the solution of the problem of utilizing tidal energy is difficult to overrate. In the materials of the International Congress on Tidal Electric Power Plants, published in 1972 [54], the small Kislaya Guba TPP as well as the industrial Rance TPP are seen as guide posts along the road toward the development of tidal power. It is also emphasized that new methods of modern construction technology were employed in the construction of the Kislaya Guba TPP. In connection with this it should be noted that the Kislaya Guba TPP was brought into operation in 1969 when, after the construction of the Rance TPP, there was an "ebbing" of interest in the problem of utilizing tidal power as a

consequence of the rejection by the French government to build the Chausey TPP. In addition, while operation of the industrial Rance TPP convincingly proved the possibility of regulating tidal power in accordance with the curve of consumption, the construction of the small Kislaya Guba experimental installation demonstrated the feasibility of greatly lowering the cost of construction by doing away with cofferdams and using the floating method to erect a tidal electric power plant.

New grandiose tidal power plant projects, drawn up in 1970-1971 in England (Severn, with a capacity of 4.5 million kilowatts) [56], Canada (Cobequid Bay, 3.5 million kilowatts, Cumberland Basin, 2.4 million kilowatts, and Shepody Bay, 2.9 million kilowatts) [50], use the Soviet floating design and even a lighter version of it. It is stated in R. Gibrat's latest paper [51] that the government of Nova Scotia, notwithstanding the deterrent of inflation, is promoting a TPP project in Minas Basin, with a capacity of 4 million kilowatts and a cost of 2 billion dollars; the urgency of carrying out this project is based primarily on the fact that a tidal power plant is a "clean" source of power which does not pollute the atmosphere and does not adversely affect fishing or water quality. /257/

Thus there is reason to believe that the Kislay Guba TPP has inaugurated the start of a new "flood" of interest in tidal power which, at the present time, is acquiring important significance as a powerful source of "clean" energy.



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Table of Contents

Preface	page 3
Chapter 1. The Kislaya Guba Tidal Power Station and the Problem of Utilizing Tidal Energy	page 7
1-1. Present Situation	page 7
1-2. Tidal Energy Resources in the USSR	page 18
Chapter 2. Description of the Construction Region	page 25
2-1. Substantiation of the Kislaya Guba Site for an Experimental TPP	page 25
2-2. Natural Conditions at Kislaya Guba	page 28
2-3. Reasons for Situating the Construction Dock at Cape Prityka. Natural Conditions	page 40
Chapter 3. Main Structures of the Kislaya Guba TPP	page 45
3-1. Layout of the Hydraulic Power System and the TPP Structure. . .	page 45
3-2. The Underwater Foundation Area Beneath the TPP Structure. . .	page 57
3-3. Connection of the Block with the Foundation, Races and Banks. .	page 64
Chapter 4. Power Engineering and Equipment	page 77
4-1. Power Engineering	page 77
4-2. Hydroelectric Power Equipment	page 76
4-3. Mechanical Equipment	page 92
4-4. The Electrical System	page 95
Chapter 5. Statistical Calculations of the Floating TPP Structure and Full-Scale Study of Its Condition Under Stress	page 104
5-1. Initial Situation	page 104
5-2. Calculations of the Static Work of the Block Longitudinally . .	page 108
5-3. Calculations of the Static Work of the Block in the Trans- verse Direction and Under Local Loads	page 115

5-4. Experimental Research	page 117
5-5. Full-Scale Static Research on the TPP Structure	page 124
5-6. Field Studies of the Stressed State of the TPP Structure as The Result of Dynamic Interactions	page 137
Chapter 6. Building Materials	page 146
6-1. Concrete of Increased Frost Resistance	page 146
6-2. Thermal Insulation and Waterproofing	page 159
6-3. Hydrophobic Soil for Filling the Voids in the Floating TPP Structure	page 172
Chapter 7. Protection of the Structure and Equipment from the Corrosive Action of Sea Water	page 176
7-1. Electrochemical Protection of the Metal Structure and Equipment from Corrosion	page 177
7-2. Electrochemical Protection of the Reinforcement of the TPP Structure from Corrosion	page 185
7-3. Anti-Fouling Coatings	page 195
Chapter 8. Special Features of the Organization of the Design, Research and Construction Work During Erection of the Kislaya Guba TPP.	page 202
8-1. Research and Design Make-up and Organization	page 202
8-2. Special Features of the Organization of Construction of the Experimental Installation	page 209
8-3. Organization of Operations at the Two Sites. Author's Supervision	page 213
Chapter 9. Concreting Operations	page 220
9-1. Choice of the Concreting Method	page 220
9-2. Erection of the TPP Structure	page 228

Chapter 10. Underwater Engineering Operations	page 241
10-1. Formation of the Underwater Foundation Pit.	page 241
10-2. Special Features of the Underwater Engineering Operations when the Floating Method of TPP Construction is Employed	page 255
10-3. Apparatus for Transmitting Vertical and Planar Positions of Points Underwater and Back Up to the Surface	page 260
10-4. An Apparatus for Levelling the Bed	page 268
10-5. Underwater Construction of the Bed Beneath the Floating TPP Structure	page 275
Chapter 11. Towage of the Floating Block	page 280
11-1. Description of the Design Solution	page 280
11-2. Flooding the Construction Area and Raising the Block	page 284
11-3. Excavation of the Gap	page 289
11-4. The Removal, Towage, Installation at the Site, Submersion and Ballasting of the Block	page 293
Chapter 12. The Hydrotechnical Significance of the Experiment of Creating the Kislaya Guba TPP	page 309
References	page 321

Lev Borisovich Bernshtein, Viktor Glebovich Gavrilov, Semen L'vovich
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