AT CONSTRUCTION PROJECTS OF THE FIVE-YEAR PLAN

ARCH DAM OF THE INGURI HYDROELECTRIC STATION*

V. I. Bronshtein, I. E. Lomov,
A. Ya. Menabde, and G. V. Rubinshtein†

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Editor's Note. The following article presents the final project decisions and changes made in the design of the Inguri arch dam upon compiling the working drawings. The indicated changes are related to refinement of the behavior of the dam under load as a result of obtaining additional data of engineering surveys and investigations. Of unquestionable interest to the journal readers are such design changes noted in the article as the replacement of the reinforced-concrete seat by an enlarged concrete seat (primarily from considerations of convenience of construction and decrease in the consumption of reinforcement), improvement of the shape of the dam, elimination of the second level of spillways, change in the scheme of the antiseismic reinforcement and construction of the seat over a fault zone, etc. In the editor's opinion, it would be useful to present in greater detail a number of problems in the dynamics of their development, in particular, an evaluation of the engineering-geologic setting of the construction site, concept of antiseismic reinforcement, and improvement of the dam shape.

INTRODUCTION

On the eve of the 61st anniversary of the Great October Revolution, the first unit of the Inguri hydroelectric station – the main step in the cascade of hydroelectric stations on the Inguri River using its drop in the lower course – was brought under an industrial load ahead of schedule. The station structures include: an arch dam, submerged tunnel-type intake, pressure diversion tunnel (with a head of more than 100 m) with a diameter of 9.5 m and length of 15 km, underground powerhouse with five vertical 260-MW units, and a 3-km-long free-flow tailrace tunnel.

The total static head is 409.5 m, of which 226 m is created by the arch dam and the remaining 183.5 m by the diversion, consisting of the pressure and the tailrace tunnels. The installed capacity is 1300 MW (the total capacity of the cascade of Inguri hydrostations is 1640 MW) and the average annual electricity is 4330 million kWh (the cascade on the whole, 5460 million kWh).

ENGINEERING-GEOLOGIC AND SEISMOTECTONIC CONDITIONS AT THE SITE

The dam is being constructed under complex engineering-geologic conditions in a region characterized by a high background seismicity (intensity 8). The foundation of the arch dam

*The contract design of the arch dam of the Inguri hydroelectric station was developed by the Moscow departments of the S. Ya. Zhuk All-Union Planning, Surveying, and Scientific-Research Institute (Gidroproekt) together with the Tbilisi division of the Institute. The working project-estimate documents are being published by the Moscow departments of Gidroproekt with the enlistment of specialized organizations (State All-Union Trust for the Stabilization of Foundations and Structures (Gidrospetsproekt), Moscow Special Design Department of Steel Hydraulic Structures (Mosgidrostal')) with the participation of the Tbilisi division of Gidroproekt.
†Deceased.

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is composed of limestones, dolomitized limestones, and dolomites of the Barremian Stage (Lower Cretaceous), the beds of which have a steep (50-60°) monoclinal dip toward the lower pool (with a small anticlinal bend in the channel part of the gorge). The foundation rocks are strong (with a compressive strength of the order of 80-90 MPa) but are severely fractured, with moduli of deformation in pressure release zones from 8-4 GPa and less and in zones of natural preservation to 13 GPa and more. Six members of rocks with a thickness from 40 to 150 m are distinguished on the basis of the character of the structure, physical and mechanical properties, and degree of jointing.

The main earthquake faults at the dam site are: the marginal dislocation at the contact of the Jurassic and Cretaceous rocks passing 1.0-1.5 km above the dam; the Ingirish thrust fault with a vertical displacement of the order of 1000 m located 1.5 km above the dam; a tectonic pinnate fracture with a vertical displacement of 100-120 m, but without signs of contemporary differential movements, associated with the Ingirish thrust fault and intersecting and right-bank abutment of the dam 110 m below its crest (right-bank fracture). Disturbances of a smaller order include about 20 large fractures with an open width of more than 10 cm, and some smaller fractures. Six main joint systems are distinguished as being developed in the rocks here. The thickness of the channel gravel alluvium at the dam site reaches 38 m. Karst phenomena occur only at the base of the high left-bank terrace, where cavern porosity and leaching of the carbonate rocks along the joints and bedding are observed.

The seismic danger in the construction region, according to the department of geophysical surveys and investigations of Gidroproekt, is determined by earthquakes which can occur in the following earthquake-generating zones of the Caucasus: in the zone of the overthrust of the Main (or Watershed) Range of the Caucassus (magnitude of maximum possible earthquake M_{max} = 6.75-7.0, depth of focus h = 20-30 km, minimum distance from dam $\Delta \approx 30$ km); in the zone of the Ingirish fault (M_{max} = 5.5, h = 5-10 km, $\Delta \approx 0$); in the region of the pinnate fracture of the overthrust of the Main Range, in the zone of the Kakur–Uskur strike-slip thrust fault (M_{max} = 6.5, h = 20 km, Δ = 25-30 km); in the zone of faults in the middle part of the Abkhaz–Svanetskii step (M_{max} = 5.5, h = 5-10 km, Δ = 10-15 km). The maximum seismic effect at the dam site can reach intensity 8. To describe the aforementioned earthquakes posing a danger to the structure, the department of geophysical surveys and investigations of Gidroproekt (I. P. Kuzin, A. I. Savich, and A. V. Suvilova) on the basis of a genetic approach constructed an ensemble of accelerograms with the most probable values of maximum accelerations 0.11-0.26 g.

The indicated values of the peak accelerations agree well with the results of calculations of the department of dynamic investigations of the research division of Gidroproekt (V. M. Lyatkher and A. D. Kaptsan) carried out by the method of statistical processing of data on earthquakes classified according to macroseismic intensity with consideration of the service life of the structure and frequency of earthquakes of the design intensity. For probabilities of not exceeding the seismic effect equal to 0.5-0.977 during the period of a controlled state of the structure τ = 100 years, the values of the maximum accelerations determined by this method and reduced to the canyon floor are within 0.08-0.20 g.

SHAPE OF THE DAM

The dam consists of an arch of double curvature with gravity abutments at the upper elevations to both banks (Fig. 1). The total construction height of the dam, 271.5 m, makes it first in the world among arch-type dams. The crest length of the dam is 758 m, including a length of the abutments of 118 m.

The design of the dam connection with the foundation at the upper elevations of the right bank, where the rocks are characterized by low moduli of deformation (of the order of 2.0 GPa) and considerable reduction of strength upon waterlogging, was selected as a result of compairing the adopted variant (a gravity abutment with cutoff wing walls and grout curtains and curtain drains inclined toward the upper pool) with alternative variants in the form of a concrete slab-mat, submerging crest (of the "Kurobe" type), and pile grillage made by the mining method. The dimensions of the abutment can be reduced with further designing, since as a result of widening the seat at the upper elevations and improving the shape of the dam, envisaged in the working drawings, the values of the forces being transmitted to the abutments by the dam will be reduced considerably.

The supporting part of the structure is made in the form of a seat separated from the arch part of the dam by a smooth perimetric joint. Compared to the technological design, the seat is considerably widened and increased in height for the purpose of reducing the pressure on the foundation, which proved to be of lower quality than in the evaluation at the stage of the contract design; the "plug," having been included as part of the seat common for the entire support contour, was eliminated as an independent design component. The height of the seat is 15-20 m in the flanks and reaches 50 m in the lower part of the gorge.

Joining the arch dam with the foundation by a seat separated from the arch by a perimetric joint made it possible: 1) to reduce the compressive stresses being transmitted to the rock mass owing to an increase in the area of the supporting surface and to differentiate them in relation to the bearing capacity of the rock; 2) due to opening of the perimetric joint equipped with special seals ensuring its water-tightness, to reduce the danger of the occurrence of tensile stresses which could arise in the contact region of the foundation as a consequence of both its inhomogeneity and seismic loads and could lead to disturbances of the grout curtain; 3) to accelerate covering the rock excavation of the foundation pit with concrete for the purpose of protecting the rock from the development of expansion due to decompression and to reduce the additional volume of rock in connection with this; 4) to smooth out the effect on the arch part of the dam of local asperities of the rock surface along the supporting contour.

To describe the geometry of the dam, an original analytic apparatus was constructed in collaboration with the Moscow Civil Engineering Institute (A. D. Dobysh) with the use of two independent functions of a polynomial form: function of the middle surface and generalized function of thickness [1]. The unknown coefficients of the polynomial expressions are determined from the condition of minimization of the weighted sum of the squares of the discrepancies between the specified values recorded graphically from the drawings of the dams and the sought functions.

Development of an analytic description of the geometry of the dam created the prerequisites for a correct conduction of calculations of its stress and strain and solution of problems of designing dam components, increased the accuracy of the solutions and reduced their laboriousness, and made it possible to completely automate by means of computer the extremely laborious work required to determine the coordinates of the construction blocks, columns, and sections. The savings just from introducing into design practice the automated system of calculating the coordinates and parameters of the overhang of the blocks in full scale amount to 50,000 rubles/yr due to a reduction in the labor intensity of the calculations.

The configuration of the perimetric joint was selected with consideration of the average (in a mean-square sense) direction of the reactions of the dam with six different combinations of loads and effects on the structure, including the hydrostatic pressure, dead weight of the concrete, and temperature and seismic effects. The perimetric joint in cross sections normal to its center line represents (Fig. 2) the arc of a circle with a radius equal to the doubled thickness of the dam at point M and with a center whose angular displacement α relative to the unit vector $\bar{t}$ of the tangent to the middle surface of the dam is selected from the condition of the passage through this center of the direction-averaged resultant of the forces $\bar{R}$ transmitted from the dam to the foundation.

The thickness of the dam in the cross section of the central cantilever (see Fig. 1) is 10 m with respect to the crest, 50 m with respect to the perimetric joint, and 90 m with respect to the contact with the foundation. The total volume of concrete in the dam, determined by the contract design, is 3,960,000 m^3 (including the gravity abutments and cutoff wing wall of the right bank). The dam is being constructed in an asymmetric site with a configuration close to parabolic and ratio of crest length to height equal to 2.3.

The shape of the dam is selected as a result of its successive improvement, beginning with a circular outline, on the basis of numerous theoretical studies and experimental investigations on elastic, brittle, and geomechanical models, in which participated, along with Gidroproekt, the All-Union Scientific-Research Institute of Hydraulic Engineering (VNIIG), research division of Gidroproekt, Georgian Scientific-Research Institute of Power and Hydraulic Structures (GruzNIIÉGS), and other organizations. The variant of the dam with arches of a five-center outline adopted in the contract design, the stress state of which was characterized by a high level of compressive stresses σ_{max}^{com} (of the order of 10 MPa under the main

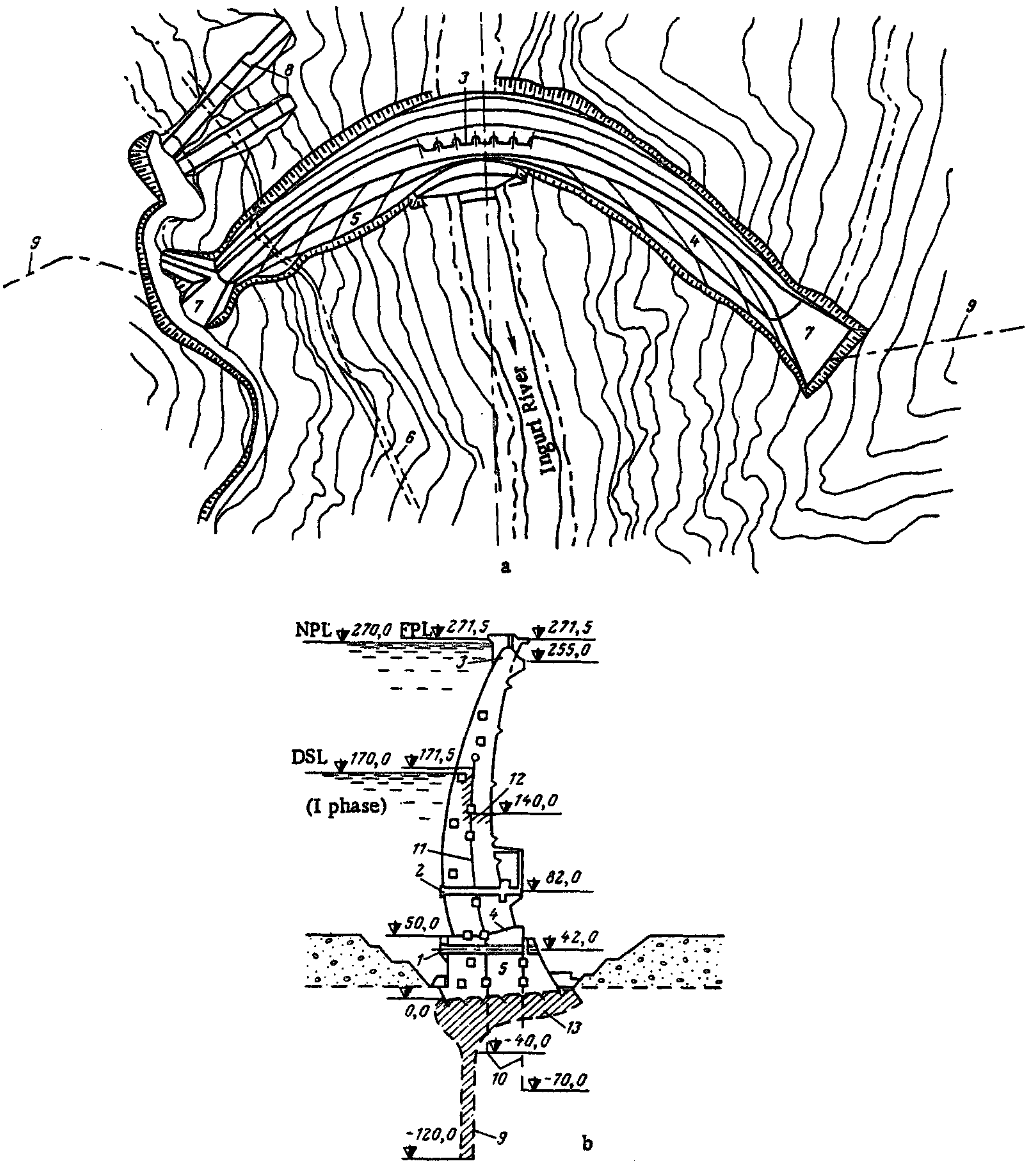

Fig. 1. Arch dam of the Inguri hydrostation. a) Plan; b) section through central cantilever. 1) Temporary outlets; 2) dewatering outlet; 3) surface spillway; 4) perimetric joint; 5) seat; 6) right-bank fault; 7) gravity abutments; 8) water intake; 9) grout curtain; 10) drainage curtains; 11) longitudinal joint; 12) boundary of first phase of dam; 13) grouting.

combination of loads and effects), smallness of the values and zones of propagation of tensile stresses, and, simultaneously, considerable nonuniformity of the distribution of stresses in the dam, continued to be improved during design. The shape of the structure was optimized by the method of successive approximations. The problem was solved by a calculation method [2], the results of which were checked on a large-scale (1:150) geomechanical model at VNIIG (S. S. Antonov and L. E. Kogan). In the first stage the middle surface of the dam was optimized for fixed values smoothed in a mean-square sense and decreased by 3% on the average relative to the thicknesses of the adopted variant, but corresponding to their former distribution over the dam. As a result a more uniform distribution of stresses in the dam compared to the adopted variant was obtained, the maximum compressive stresses were reduced by as much as 1 MPa, and the volume of concrete was reduced by about 100,000 m^3. The middle sur-

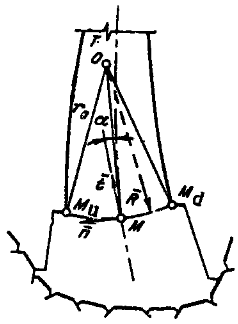

Fig. 2. Diagram showing construction of the cross section of the perimetric joint. O and r_0) Respectively, the center and radius of circle M_uMM_d; n) unit vector of the normal to the middle surface at point M; $\bar{t}$) unit vector of the tangent to the middle surface at point M; $\bar{R}$) vector of the resultant of the forces of the dam at point M; α) angle between vector $\bar{t}$ and straight line OM.

face of the dam in the working drawings is characterized by the absence of discontinuities in the curvatures and their related stress concentrations, which are inevitable in dams, for describing the arch elements of which multicenter curves are used, and also by rigorous regularity of both horizontal and vertical curvatures.

The decrease in the maximum tensile stresses obtained made it possible to undertake a reduction in the thicknesses of the dam while preserving the level of stresses corresponding to the approved contract design. In solving this problem under conditions of a high rate of dam construction it was required to take into account the following circumstances: 1) the geometric parameters of the dam can be varied within limits such that the changes in the shape of the structure would not be a cause for delay of design and construction; 2) a smooth joining (to the second derivative inclusively) of the lower part of the dam, poured according to the geometry of the refined contract design, with its upper part should be provided; 3) the concrete can be trimmed away only on the downstream face, since alteration of the upstream face would require changes in the designs of the inclined gate seats, arrangement of the cutoffs and drains, galleries, and shafts; 4) the new geometry of the dam should have an analytic description.

Calculations, which were confirmed by testing the geomechanical model, substantiated a form of trimming which with a 50,000-m^3 reduction in the volume of the dam (mainly within the upper third of the structure) provided minimum changes in the stress state relative to the initial variant with an optimized middle surface (not more than 0.5-0.6 MPa). The trimming scheme is shown in Fig. 3. Over the height of the structure trimming increases from zero at a certain specified elevation z_0 to a maximum, after which it decreases to zero at the crest; in horizontal sections it increases from the crown to the perimetric joint. The maximum amount of trimming is 2.6 m. In the final variant of the dam geometry the arches have a variable thickness insignificantly increasing from the crown toward the spring lines in the central zone of the structure and markedly increasing near the abutments, and gradually changing variable curvatures with maxima at the crown and the arch spring lines.

STATE OF STRESS AND STRAIN

The state of stress and strain in the final variant of the dam geometry under the main combination of loads and effects (hydrostatic pressure at the full reservoir level of 270 m and headwater level of 210 m, dead weight of the concrete, and temperature-effects) was substantiated by calculations of the dam by an automated trial load method [3] and by investigations of geomechanical [4] and temperature [5] models. The results of the calculations and experimental investigations agree satisfactorily. The stress field of the dam is characterized by good uniformity, high degree of use of the compressive strength of the concrete (the maximum compressive stress is 9.4 MPa), and practical absence of tensile stresses in the arch part of the dam.

To evaluate the state of stress and strain in the structure under seismic effects, calculations were made according to the linear spectral-dynamic theory by the trial load method [6] and method of isoparametric finite elements (main computer center of Gidroproekt), investigations of brittle models were conducted in a harmonic regime, bringing them to failure on a seismic platform [7], experimental analytic investigations (VNIIG) were carried out with loading of large-scale models located on a VP-100 vibrating platform by means of a pulsed source and with computer calculations of stresses corresponding to the given earthquake accelerogram, and calculations were made by a research division of Gidroproekt based on the solution of complete equations of thin shells by the finite-difference method for the seismic effect given by the accelerogram [8]. In these last calculations three components of the seismic effect, its change along the support contour, and the possibility of opening of the intersectional, block, and perimetric joints of the dam were taken into account.

DESIGN ELEMENTS OF THE DAM

The dam is cut into 38 sections by helicoidal intersectional joints normal in any horizontal section of the axis of the corresponding arch. The axes of the joints are being constructed on the middle surface of the dam at a distance of about 16 m apart. In the channel part of the seat the sections are constructed as three columns, in the arch part to an elevation of 176.0 m the sections are being constructed as two columns, and higher as one column. The longitudinal joint dividing the sections of the dam into upstream and downstream columns is being made approximately along its middle surface. Concerted work on individual sections and columns of the structure is provided by vertical offsetting and grouting of the intersectional joints and longitudinal joint.

Primary grouting of the joints is being performed at sealing temperatures established on the basis of an analysis of the overall stress in the dam according to the criterion of the technical and economic expediency of cooling the concrete masonry to various sealing temperatures for the provision of the opening width of the joints necessary for grouting: the seat and arch to an elevation of 120.0 m are being sealed at a concrete temperature of 10°C and the arch above the 120.0 m mark, 12°. The design of the grouting outlets permits repeated and multiple grouting of the dam joints. For grouting the joints during construction and their regrouting during operation of the structure, for collecting and draining seepage, observing the operation of the drains, checking the condition of the structure and performing repair work, communicating inside the dam, laying supply lines, installing the switchgear for the control and measuring equipment, and for other purposes, in the dam are being constructed five levels of service galleries located about every 50 m over the height of the structure and continued as grouting and drainage tunnels in the walls of the gorge, and also several additional galleries, including perimetric galleries. Each level represents a system of longitudinal and transverse galleries having an exit to catwalks, which on the downstream face are arranged every 20-25 m over the dam height. The construction of freight elevators and stairways is envisaged for communicating between the levels of the galleries and dam crest.

Watertight seals in the form of brass sheets with a loop expansion piece are being installed in the dam joints. The number and width of the seals are determined on the basis of the permissible gradient of the head during seepage in the concrete, assumed equal to 40, and are: for heads from 200 m and more, three rows of seals of width 1360 mm; heads of 100-200 m, two rows of width 1360 mm; for heads of 100 m and less, two rows of width 687 mm.

The drainage system underwent substantial changes from the contract design. In the lower two-thirds of the structure drainage is accomplished by means of horizontal drains being placed on top of the concreted blocks every 1.5 m over the dam height and being extended into drain wells located in the intersectional joints. In the upper third of the dam drainage is accomplished in the form of 105-mm diameter dead-end wells bored from the longitudinal or transverse galleries. The dam is being constructed in two phases (Fig. 1).

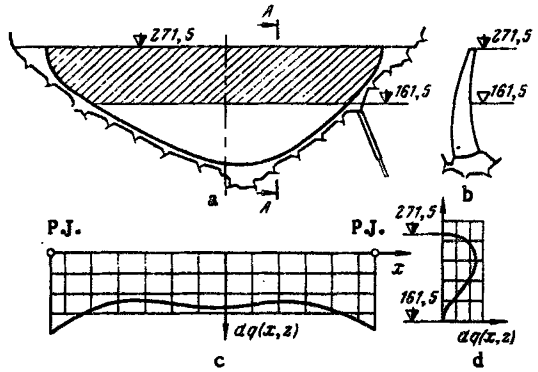

Fig. 3. Diagram of trimming concrete on the downstream face of the dam. a) View from downstream face; b) section through A-A; c and d) graphs of the change in thickness respectively in horizontal and vertical sections; the region of trimming is hatched. (P.J. = perimetric joint).

REINFORCEMENT

The contract design called for reinforcing the faces of the dam, perimetric joint, seat, galleries, elevator shafts, drain wells, and also special antiseismic (earthquake-proof) reinforcement with a total consumption of reinforcement steel of about 70,000 tons and specific consumption per m^3 of concrete of about 17.5 kg. In this case the quantity of reinforcement was determined from the condition of the reinforcement absorbing the total tensile stresses in the concrete occurring in the calculated sections under the worst combinations of the design loads and effects with consideration of stress concentration around openings, with the exception of those cases where the tensile forces did not pose a danger to the strength and longevity of the dam or its elements.

In the detail design additional experimental studies and refinement of the calculation schemes and load made it possible to change from the principle of reinforcement based on total tensile forces to dispersed reinforcement [9], which increases the tensile strength of the concrete. The dispersely reinforced dam in zones of occurrence of tensile stresses behaves as a practically monolithic concrete structure. Vertical reinforcement and reinforcement of the sections of the structure in a horizontal (arch) direction are being performed by the dispersed scheme. In the intersectional joints the amount of horizontal reinforcement was selected from the condition of its absorbing the total tensile force, since the joints are not under tension.

The contract design called for a reinforced-concrete construction of the seat with an average consumption of 40 kg of reinforcement per m^3 of concrete. In the detail design, on the basis of additional design studies and an analysis of the results of investigations on the geomechanical model, it was possible to practically eliminate reinforcement of the seat, with the exception of two small zones near the downstream face of the structure, where there are considerable tensile stresses directed along the support contour, and several local zones located on severely disturbed and inhomogeneous parts of the foundation, where marked breaks in the supporting surface were formed by final clearing of the rock. Reinforcement of the downstream face of the seat, surfaces of the perimetric joint, and zones around the galleries and wells is also stipulated. The specific consumption of reinforcement per m^3 of concrete of the seat was reduced to 6.5 kg, or by 11,500 tons in all.

The average consumption of reinforcement in the first phase of the dam was 8.2 kg/m^3, in which case a considerable part of this reinforcement is for reinforcing the concrete around the galleries and shafts and also went into the precast reinforced concrete of the balconies, formwork of the galleries, and intersectional joints.

For approximating the elastic character of deformation of the dam under seismic effects of the design intensity, an original design of horizontal reinforcement in the upper quarter of the structure with passage of the reinforcement through the intersectional joints was developed. The established notion about an arch being only under compression was the cause of the unfounded objections to the design reinforcement of individual zones of such a dam occurring in an extended arch. Upon a substantial drawdown of the reservoir of the Nurek hydrostation (to 90 m) its upper zone becomes practically uncompressed. In this case the intertial forces under a seismic effect cause in it a substantial eccentric tension, which cannot be absorbed without introducing the design reinforcement.

As experimental investigations [14] showed, in the absence of such reinforcement the vibrations of the dam are accompanied by considerable openings of the intersectional joints and formation of horizontal through cracks in sections of the dam under the effect of accelerations considerably less than the design. Such a state is impermissible. The adopted scheme of antiseismic reinforcement made it possible to improve the operating properties of the dam with a simultaneous reduction of antiseismic reinforcement by 10,000 tons relative to the contract design.

The reliability of the dam, along with the special choice of its shape and antiseismic reinforcement, is provided by the construction of seals of increased reliability in the intersectional and perimetric joints, protection of the contact of the seat with the rock in the lower part of the gorge on the upstream side by bitumen mastic, coating of the upstream face in the lower third of the dam with rubber-epoxy waterproofing, installation of a system for repeated and multiple grouting in the dam concrete, construction of a developed drainage system and tunnel-gallery system in the walls of the gorge and in the dam, which, if necessary, permit conducting repair work, and also engineering measures on consolidation of the foundation.

FOUNDATION STABILIZATION

Preparing the underground contour of the dam includes stabilizing grouting of the foundation to a depth of 30 m with a design volume of 250,000 m^3, a grout curtain with a total length in plan of 1106 m and depth of 120 m, being made from six levels of the grouting galleries in descending and ascending fragments, a curtain drain with a depth from 70 m in the channel part of the gorge to 150 m in the gorge walls.

The stability of the dam is substantiated by calculations by the method of limit equilibrium of the bank supports with the required safety factor of 1.8 and by calculations of the stress in the foundation, which is regarded as an elastoplastic medium, by the finite-element method [10, 11]. The calculations established the need for sealing off with concrete the right-bank fault and five large tectonic fractures with a width of 25-30 cm and more filled mainly with clay and clayey loam with calculated parameters of shear strength $\tan\varphi = 0.65$ and $c = 0.05$ MPa. Sealing of the fault (Fig. 4) is being performed by the mining method in the form of a continuous concrete mass about 10 m thick in its upper part adjacent to the dam (within elevations 150.0-115.0 m) and in the form of 10×10 m columns in the lower part of the dam (within elevations 115.0-75.0 m). Such a design provides a gradual change in the rigidity of sealing and thereby makes it possible to reduce the stress concentration and distortion of the seepage flow in the surrounding rock. The total length of sealing is 150 m, depth 75 m, and volume of concrete in the continuous part 22,000 m^3 and in the column part 12,000 m^3.* To stabilize the rock mass in the zone of sealing the fault and fractures and to eliminate the unfavorable effect of pressure release while driving the tunnels, grouting to a length of 100,000 linear meters, which links with the grouting of the dam foundation and grout curtain, is stipulated. Primarily continuous concreting of the joints is called for, the blocks of the rock mass of the foundation in the peripheral zone, where the stresses are small, being left as they are. The volume of sealing the fractures is 42,000 m^3 of concrete.

Calculations of the foundation also established regions of plastic deformation of the rock mass near the downstream face of the dam at lower elevations at places of a shallow in-

*Here and henceforth the design volumes of stabilizing measures in the foundation are given. The actual volumes were increased by a factor of about 1.5.

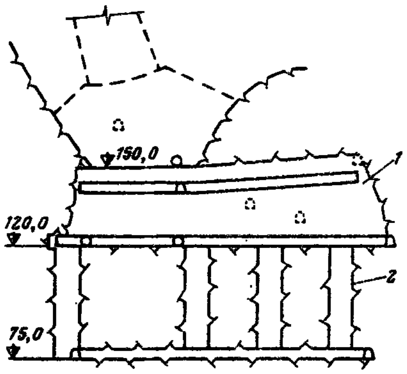

Fig. 4. Sealing of the right-bank fault. 1) Continuous sealing with concrete; 2) column sealing.

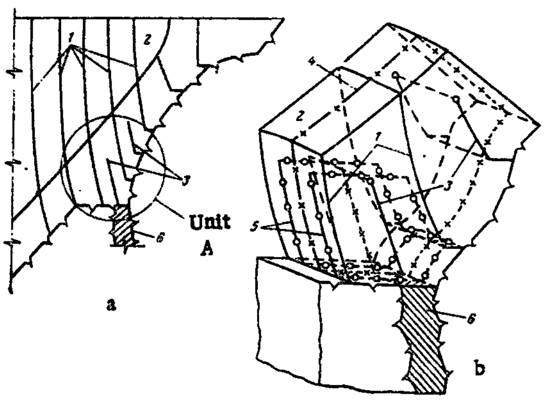

Fig. 5. Design of the seat over the fault zone. a) Right-bank abutment of the dam; b) enlargement of unit A. 1) Intersectional joints; 2) perimetric joint; 3) additional joints oriented along the fault; 4) longitudinal joint; 5) additional joints oriented parallel to the middle surface; 6) fault.

cision. The formation of plastic regions in the foundation is associated with a local increase in its deformation, which unfavorably affects the stress in the dam. To neutralize this phenomenon, stabilization measures were planned in the lower pool at low elevations of both banks in the form of a concrete slab and installation of prestressed anchors.

A special design of the seat over the fault zone [12], shown in Fig. 5, was developed to compensate for possible small near-surface differential displacements in the zone of the right-bank fault, the occurrence of which can be provoked by the construction of the dam and formation of a reservoir storing large masses of water. The height of the seat over the fault was increased to its double thickness and the seat itself was cut by two systems of smooth joints allowing some freedom of movements of the foundation. The joints of the first system are oriented approximately along the fault, some are joined with the intersectional joints and others are located within the section and cut the seat from the rock surface to half its height. The joints of the second system are oriented parallel to the middle surface and reach to half the height of the seat. As the investigations of VNIIG [13] showed, such a design of the seat will absorb movements up to 10 cm with practically no change in the state of stress in the dam.

FLOOD CONTROL

In the dam, 182.5 m below the crest, there are seven 5-m diameter dewatering outlets, four of which are equipped as idle spillways that can pass a maximum construction-period discharge of 1% probability (1200 m^3/sec) at heads greater than 25.0 m and a discharge of 0.1% probability (1860 m^3/sec) at the highest flood level 271.5 m, and nappe deflectors. After construction of the Inguri pumped-storage station, envisaged in the future, five of these seven outlets will be used for delivering water to the units of that station and two will be left as spillways in the dam.

For the dam's spillways Mosgidrostal' developed unique mechanical equipment which is intended to absorb the static head and regulate discharges at heads up to 181 m. This development made it possible to dispense with the second level of outlets called for by the contract design.

The main regulating and guard gates as designed are made as slide gates. The special outline of the lower edge developed for the regulating gate provides compactness of the water jet for any openings and absence of cavitation on its downstream side. The design of the ring follower guard gate permits a considerable reduction in the dynamic loads on the dam. For sealing the gates polyethylene seals, developed by Mosgidrostal', were used for the first time and they provide a high strength and lower coefficient of friction at smaller overall dimensions compared to the traditional rubber seals. The elimination of the two-level design of dewatering outlets reduced the cost of the dam by about 6 million rubles.

On the dam crest is a surface spillway (six 9-m bays) designed to discharge 1200 m^3/sec at the full reservoir level. On boosting the water level by 1.5 m (to the highest flood level) the spillway can discharge a maximum design flow with a peak of 2500 m^3/sec (0.01% probability with a guaranteed correction). In this case the maximum flow over the surface spillway reaches 1800 m^3/sec.

For discharging water during the period between closing the temporary diversion tunnel and start-up of the dewatering outlets, three temporary 3×5-m outlets are constructed in the plug of the dam, which are intended for passing the winter flood with a discharge up to 400 m^3/sec. At the present these outlets are closed on the upstream side by vertical lift gates calculated to resist heads up to 160 m. After starting up all units of the hydrostation in 1979-1980 the outlets are to be sealed with concrete.

To protect the river channel and banks from scouring during operation of the flood-control works, a stilling basin is provided for in the lower pool. To lighten the apron of the stilling basin it is made in the form of an inverted arch supported on the concrete bank masses.

ON-SITE OBSERVATIONS

Control and measuring equipment consisting of 7000 instruments is being installed in the dam and its foundation for routine monitoring of stresses and strains, seepage and temperature regimes, displacements, operation of the flood-control works, conditions in the lower pool, and long-term investigations of seismic occurrences and tectonic movements of the earth's crust, starting from completion of dam construction and filling the reservoir. By the time of start-up 1278 instruments had been installed, the operating failure of which was 11.3%.

MAIN STAGES OF CONSTRUCTION

Construction of arch dam began in 1965 with driving of the 560-m diversion tunnel at a cross section of 130 m^2 on the left bank. In 1969 construction of the tunnel was completed, closure of the Inguri River took place, and work was launched on excavating the foundation pit, which had started in 1967. The depth of the cut was determined by calculations of the stability and strength of the bank supports with consideration of the thickness of the zone of weathering and pressure release of the wall rock, and on the average was 35-40 m in the walls and about 50 m in the channel. The pit was excavated in benches in descending order by the smooth blasting method with hauling of the rock along intermediate roads on the gorge walls. The total (profile) volume of the cut is 2.6 million m^3, including 1.7 million m^3 of rock.

The foundation was prepared for pouring concrete in three stages. At first rough cutting was carried out to the design depth, then on the basis of geological data and special geophysical investigations additonal rock was removed, and final cutting and clearing was performed to obtain a rock surface of the necessary quality (a propagation velocity of longitudinal seismic waves $c_V \geq 1$ km/sec).

The first concrete was poured in the left-bank seat in 1971. In the first phase of the dam's construction concrete was placed in the following volumes by years (thousand m^3): 48 in 1972; 185 in 1973; 268 in 1974; 332 in 1975; 350 in 1976; 450 in 1977; and 530 in 1978. Concrete was delivered to the 1.5-m-high blocks by four cableways with a span of 934 m and capacity of 25 tons each. Concrete grade M350, impermeability V12, the main composition of which (composition 3) was awarded the State Seal of Quality in 1978, is prepared at a continuous batching plant with an output of 300 m^3/h and at a cyclic plant with an output of 100 m^3/h.

Construction is being accomplished in two phases. The first phase (Fig. 1b), which led to start-up of the first units at a headwater level of 170.0 m, was constructed to an elevation of 171.5 m with respect to the upstream columns and to 140.0-161.0 m with respect to the downstream columns. The upstream columns were sealed to the 161.0-164.0 m level, and the downstream columns, including grouting of the longitudinal joint, to 120.0 m.

The state of stress of the dam's first phase on the basis of the calculations of Gidroproekt and model investigations of VNIIG is characterized as follows: the compressive stresses do not exceed 4.0 MPa in the cantilever direction and 2.3 MPa in the arch direction. The small cantilever tensile stresses (up to 0.7 MPa), obtained according to the calculation in the central part of the upstream face, upon building up the dam and a rise of the water level to the full reservoir level will change into compressive. The design prediction of the state of stress in the dam at the first phase was confirmed by data of strain-gauge measurements in the structure obtained by GruzNIIÉGS after the water rose to the start-up elevation of 170.0 m.

In December 1977 the first phase of the 210-m-long stilling basin was completed, after which the Inguri River was directed through the temporary outlets, and the temporary tunnel was closed and sealed with a concrete plug. In April 1978 the gates on the temporary outlets were lowered, the gates of the service spillways were tested, and the first stage of filling the reservoir (to depths of 60-65 m) was carried out. During subsequent filling to the start-up elevation the level was drawn down by a depth of more than 10 m three times.

In October 1978 the dam and its underground part were completed to the extent of the start-up minimum (2,087,000 m^3 of concrete had been placed by Nov. 1, 1978). At the start of November the reservoir level reached the start-up elevation and on November 5 the first unit of the underground hydroelectric station was connected into the integrated power system of Transcaucasia. Before the end of 1978 another two units were put into industrial operation, after which the installed capacity of the main step of the cascade of Inguri hydroelectric stations reached 780 MW. By the end of August 1979 the Inguri hydrostation produced the first billion kWh. In 1979-1980 it is planned to start up the fourth and fifth units, after which the hydrostation will reach the design capacity of 1,300,000 kW. It is planned to complete construction in 1982.

LITERATURE CITED

1. A. D. Dobysh, "Concerning the assignment of the geometric shape of arch dams," in: Mathematical Programming and Calculation of Structural Elements [in Russian], MISI im. Kuibysheva, No. 83 (1970).
2. V. I. Bronshtein, "Problem of designing equally strong arch dams of minimum weight," in: Proc. of the Confs. and Meetings on Hydr. Eng.: Limit States of Hydraulic Structures [in Russian], Énergiya, Leningrad, No. 119 (1978).
3. V. F. Ivanishchev and V. I. Bronshtein, "Development of a calculation procedure for arch dams by the trial load method with the use of a digital computer," Tr. Gidroproekta, Moscow, No. 28 (1973).
4. S. S. Antonov and L. E. Kogan, "Investigation of engineering measures on stabilizing the foundation of a geomechanical model of the Inguri dam," Izv. VNIIG, Énergiya, Leningrad, 124.
5. G. V. Bedukadze and G. I. Chilingarishvili, "Investigations on strength models of the stress state in arch dams from a change in the ambient temperature during operation,"

in: Trans. of Joint Confs. on Hydr. Eng.: Investigations of the Thermics of Concrete and Reinforced-Concrete Hydraulic Structures [in Russian], Énergiya, Leningrad, No. 103 (1975).
6. V. F. Ivanishchev, N. A. Kolobova, and A. I. Potapova, "Calculations of the seismic stability of the arch dam of the Inguri hydroelectric station with the use of the trial load method," in: Trans. of the Joint Conf. on Hydr. Eng.: Seismic Stability of Large Dams [in Russian], Énergiya, Leningrad, No. 87 (1973).
7. Sh. G. Napetvaridze et al., "Investigation on models of the character of failure of the concrete arch dam of the Inguri hydroelectric station under a seismic effect," in: Coll. of Sci. and Tech. Articles of GruzNIIÉS [in Russian], Énergiya, Moscow, No. 2 (57) (1974).
8. V. N. Lombardo, "Method of calculating the stress state in arch dams for given displacements of the support contour," in: Improvement of Methods of Calculation and Design of Hydraulic Structures Being Constructed in Seismic Areas [in Russian], Énergiya, Leningrad (1976).
9. M. A. Zubritskaya, A. E. Minarskii, and I. B. Sokolov, "Dispersed reinforcement of massive hydraulic structures," Gidrotekh. Stroit., No. 6 (1979).
10. S. I. Taicher and Yu. B. Mgalobelov, Calculations of the Stability of Rock Bank Supports of Arch Dams [in Russian], Énergiya, Moscow (1972).
11. Yu. B. Mgalobelov, "Investigation of the bank supports of arch dams with consideration of elastoplastic deformations," Tr. Gidroproekta, Moscow, Coll. 33 (1974).
12. S. S. Antonov, L. E. Kogan, I. E. Lomov, et al., Author's Certificate No. 571542 (USSR), "Arch dam for seismic areas with tectonic fractures," Byul. Izobret., No. 33 (1977).
13. S. S. Antonov and L. E. Kogan, "Investigation of a new arch dam design," Izv. VNIIG, Énergiya, Leningrad, 116.
14. P. A. Gutidze and D. I. Dzhishkariani, "Model investigation of the effect of reinforcement on cracking in an arch dam under a seismic effect," in: Coll. of Sci. and Tech. Articles of GruzNIIÉGS, Énergiya, Moscow, No. 3 (58) (1976).