ERTAN HYDROELECTRIC PROJECT

(YALONG RIVER)

雅砻江
二滩工程

REPORT OF UNITED STATES TEAM'S VISIT
TO PEOPLES REPUBLIC OF CHINA
MARCH 21 THROUGH APRIL 11, 1983

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F O R E W O R D

The members of the second U S Team of specialists wish to convey to their Chinese hosts sincere appreciation for the warm reception and gracious care during the visit. The team found their Chinese counterparts to be knowledgeable, skilled professionals.

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INTRODUCTION

I. Background

The Government of the United States of America (US) and the Government of the Peoples Republic of China (PRC) signed a hydroelectric protocol in Beijing, China, on August 28, 1980, for the purpose of establishing and promoting cooperation in the field of hydroelectric power and related water resource management. This protocol is pursuant and subject to the agreement between the two governments on cooperation in science and technology signed in Washington, D.C., United States, on January 31, 1979.

On March 15, 1980, during the visit of a US delegation to China, members of the US and PRC delegations signed Annex I to the Hydropower Protocol. Paragraph 5 of Annex I is concerned with cooperative activities on the Ertan Hydroelectric Power Project in China and is divided into three main parts and two subparts.

Part 5.1

As requested by the PRC Ministry of Electric Power, the US Water and Power Resources Service* will send a team of five specialists to China for 3 months from August to October 1980, to study jointly with Chinese specialists the design alternatives for the Ertan Hydroelectric Power Project, including navigation feasibility, and to make a study tour to the damsite.

Subpart 5.1.1

If the above joint study discloses the need for further work and the Ministry of Electric Power desires further assistance, the details of such assistance will be negotiated separately between the Ministry of Electric Power and the Water and Power Resources Service.

Subpart 5.1.2

During the construction phase of the Ertan Project, the Water and Power Resources Service will provide onsite assistance in construction engineering and management if desired by the Ministry of Electric Power. The details of such construction assistance will be negotiated separately between the Ministry of Electric Power and the Water and Power Resources Service 6 months prior to the beginning of construction.

Part 5.2

As requested by the Water and Power Resources Service, the Ministry of Electric Power will send five specialists to the United States for 1 month to visit the Engineering and Research Center of the Water and Power Resources Service and other facilities of relevance to this project.

*It should be noted that the US Water and Power Resources Service has been renamed the US Bureau of Reclamation (USBR) and will be referred to as the USBR in the following text.

Part 5.3

The Ministry of Electric Power and the Water and Power Resources Service will hold further discussions after these visits and may decide on further cooperation on the comprehensive development of the Ertan Project and related scientific and technical research.

Part 5.1 provided a team of five specialists would visit China for 3 months. By agreement with the Ministry of Electric Power, it was determined that two visits of approximately 6 weeks each would be more productive than one 3-month visit. The first visit took place from October 8 through November 14, 1981.

The second six-man team visited the PRC from March 21 through April 11, 1983. See Appendix B for makeup of the team. Their itinerary is given in Appendix C.

This report covers the visit of the second group of US specialists (herein referred to as "the team") and their review and suggestions on the Ertan Hydroelectric Power Project feasibility study. The study is being conducted by the Chengdu Hydroelectric Power Survey and Design Institute (CHPSADI), Chengdu, China. This Institute is the principal coordinator and design group for the project.

The activities under part 5.2 took place in early 1981.

II. Objective of Visit

The specific objective of this visit was to review and comment on the Ertan Project feasibility study prepared by CHPSADI. The feasibility study recommends the alternative layout of a double-curvature arch dam and underground powerplant. The flood discharge structures are mainly composed of overflow spillways and middle level outlets. In addition, there are ski jump chute spillways on either side of the control overflow spillway and a tunnel spillway in the right abutment, all used as auxiliary spillways. All these spillways have flip buckets.

The team focused on the preceding proposal and alternatives thereto. As an adjunct to the objective of reviewing the feasibility study, a second objective of the visit was to display to the PRC Government that in addition to the US Government engineering agencies, the United States also has world-renowned engineering talent in the commercial (non-Government) sector.

III. Process

The members of the team began their preparation for the visit by reviewing the January 1982 report of the first team. In addition, a list of 24 specific questions relative to Ertan Project (see Appendix E) was forwarded for review. Upon arrival in Beijing, discussions were held to confirm the schedule and objective. The team reviewed a variety of hydraulic, geologic, and structural models of various project components in Beijing, Tianjin, and Chengdu. CHPSADI provided a brief summary (translated to English) of their feasibility study results in Chengdu. Photo copies of four drawings from the summary are included as Appendix F. The drawings illustrate the concepts of the dam and powerplant. Numerous documents in Chinese were interpreted by team members Yeh and Chiang. Following technical briefings and site visits, individual technical discussions were held on the functional areas of hydraulics, geology, structures, and construction management. Group discussions involving all specialties were held on: 1) structural stability of the dam and foundation; 2) the configuration of the spillway; and 3) the configuration of the underground powerplant. Each of the specific questions relative to Ertan Project were discussed during the individual technical discussions. The substance of answers to these questions is included in this report. It was recognized that complete answers to many of the questions would require more time and study than this visit permitted.

The team met regularly at meals and in the evenings to discuss their reviews. A final presentation of comments, conclusions, and recommendations was made by the team in Chengdu on April 8, 1983.

A listing of Chinese engineers and specialists who participated in site visits and technical discussions in Dukou is included as Appendix A.

CONCLUSIONS AND RECOMMENDATIONS

The review time available for the team was limited for such a complex project. Thus, the team focused on concepts, approaches to problems, and methods of analysis. The team is of the opinion that:

1. The appropriate design considerations for the project are being properly identified and evaluated.

2. With the proper implementation of design principle and construction techniques discussed, the proposed plan is technically feasible.

Although the team believes the plan is technically feasible, it also believes some improvements may be possible and should be considered in the final design. The more significant ones are:

1. Possible reconfiguration of the spillway system to eliminate the ski jump chutes because of cost and construction difficulties.

2. Possible reconfiguration of the underground powerplant complex to eliminate the underground transformer chamber and modify the surge chamber to reduce construction difficulties and minimize reliability and safety risks in the transformer chamber.

The team concurs that two potential landslides exist on Jin Long Mountain near the construction site. A massive deep-seated slide with severe complications has a very low probability of occurrence. Shallower slides at the same location are thought to have a fairly high probability of occurrence as the reservoir is raised. Impacts from slides would involve the buildings on the mountain and the diversion tunnels. The lesser slide should not severely impact the dam or reservoir.

The team believes the Ministry of Water Resources and Electric Power should consider:

1. A detailed review of final design by engineers experienced with high dams and large hydroelectric installations.

2. Continuous consultation in the field of geology beginning in the design phase through all dam foundation and major underground excavation.

3. Continuous consultation in the field of construction management beginning in the design phase through all construction.

The Ministry may wish to consider formation of a consulting board to assist management through the design and construction period.

The team compliments all who have contributed to the extensive studies and investigations. The quality of the work is indicative of skilled professionals.

POTENTIAL ENGINEERING SERVICES

Based on observations of the team, following are potential professional services which could be provided by US companies or agencies:

- Review of final designs
- Provide specialists for on-site work and/or participation on consulting board

ENGINEERING GEOLOGY

I. Observation of Geologic Model

March 28, 1983, Chengdu Institute of Science and Technology. - We viewed an excellent three-dimensional geologic model of the dam which was superimposed on the geologic formations in the foundation enabling the team to obtain a better understanding of the important areas in the right abutment.

II. Introduction to Engineering Geology

The engineering geologic work done on this project to date is outstanding and the coverage was very complete. There have been a large number of adits dug into each abutment of the damsite, and into the locations of the proposed underground chambers. Adits have also been dug into Jin Long Mountain to investigate possible landslide areas. These adits have been invaluable to CHPSADI geologists and designers and to the US team in obtaining first-hand knowledge about underground rock characteristics in areas of important structures.

The adits also made it possible for CHPSADI to perform comprehensive in situ testing of rock properties and to obtain samples of rock for laboratory testing. In addition, a large number of exploratory holes have been drilled and other investigative work performed.

There have been excellent presentations by CHPSADI of findings from the investigations, testing, and analyses. Summaries of the data were provided the team, and these were relied on in formulating conclusions presented in this report. No original geologic data were furnished or reviewed.

The January 1982 report of the first US team describing its visit of October 8 to November 14, 1981, provides an excellent summary description of geologic conditions which will not be repeated herein. Recommendations of that report have been followed and in some cases new, important data have been gathered which may result in slight modifications of conclusions.

III. General Rock Description

A detailed description of the stratigraphy and lithology is contained in the first US team report. The most important rocks at the damsite are: 1) syenite, an intrusive rock which occupies most of the dam's left abutment and the underground powerplant areas; 2) basalt, of several categories which occupies most of the right abutment and other areas not intruded by syenite dikes; and 3) uralitized basalt, a problem rock in two zones of the right abutment. There are well-developed joint systems in all of the rock, which have been mapped in detail. There are several minor faults including the most important one designated F-20 located in the right abutment dam foundation. The faults appear inactive and are quite thin. They should have little influence on the design or construction of the dam.

All of the unweathered rock at the site is of excellent quality and appears to be water tight. The weathered rock is much softer. The uralitized basalt poses a difficult problem for the right abutment foundation for the dam.

One of the important rock characteristics for design of the dam foundation, including abutments, and for the underground chambers is that of "modulus of deformation." The first US team presented suggestions on design values for each important category of rock, based on data collected before that time. Since then, a considerable number of excellent large-scale tests have been performed in the adits, and summaries of results furnished to the team. The portions of these summaries covering "mean" and "low mean" values from in situ tests of the rock on which the dam is intended to rest are presented in table 1.

TABLE 1

MODULUS OF DEFORMATION
SUMMARY OF FIELD TEST DATA

Rock Type	Rock Condition First USBR Team System)	Rock Condition (CHPSADI)	Number of Tests	Mean Value 10^3 kg/cm^2	Low Mean Value 10^3 kg/cm^2
Syenite	Unweathered	Best	9	431	340
	Slightly weathered	Medium	12	154	115
Altered Basalt	Unweathered	Best	10	531	444
	Slightly weathered	Medium	8	121	86
Augen Basalt	Unweathered	Best	6	361	292
	Slightly weathered	Medium	4	119	93
Uralitized Basalt		Best	6	328	287
		Medium	18	119	83
		Low strength	10	37	27
		Weak	23	10	6

A second important rock characteristic for design of the dam foundation is that of shearing strength, particularly for the zones of rock immediately downstream of the dam. Additional tests of this characteristic have also been performed, with the results shown in table 2.

TABLE 2

IN SITU SHEAR TEST RESULTS

Rock Type	Rock Condition	Friction Coefficient (tan ϕ)	Cohesion kg/cm^2
Syenite	Intact	2.2	30
	Joint	0.7	0
	Without normal load	--	Avg. = 46 Low Mean = 34
Altered Basalt	Intact	1.65	6
	Without normal load	--	Avg. = 16
Uralitized Basalt	Intact	1.9	21
	Intact	1.52	13
Augen Basalt	Intact	1.82	30
	Without normal load	--	34

IV. Dam Foundation

Selection of appropriate criteria for use in the design calculations is strongly influenced by a knowledge of procedures and shortcomings of the field and laboratory tests of rock specimens. In general, laboratory tests give more reliable results for the intact rock than field tests, because good samples are possible to get and better control can be exercised. The opposite is true for jointed rock. Large-scale field tests for rock which is jointed, or even minutely fractured, are more reliable than laboratory tests. These latter tests must be, and were, performed in the adits. Both types of test, and other tests, are influenced to some extent by general relaxation of rock due to stress relief around the adits, and thus tend to give values which are lower than that possessed by the rock in-place and not stress-relieved.

With respect to the "modulus of deformation" values used in the analyses of the dam and foundation, the team suggests using the weighted "low mean" averages, although the team believes this is well on the safe side. Probably the most accurate representation would be the weighted "mean" value.

For analyzing the stresses and deformations in the dam foundation by either the finite element or the trial load methods, it is suggested that the following values for Deformation Modulus be considered for the foundation rock.

Syenite	200×10^3 kg/cm^2
Basalt	100×10^3 kg/cm^2
Confined Uralitized Basalt	25×10^3 kg/cm^2
Unconfined Uralitized Basalt	6×10^3 kg/cm^2

These suggested values are more generalized than have been used in analyses so far, but we doubt that more refinement is justified. It is assumed that the foundation will be excavated to sound rock (unweathered to slightly weathered), at least 50 meters deep horizontally. Any loosened rock immediately in contact with the foundation concrete should be removed or pinned with post-tensioned rock bolts for temporary stability. Slabs loosened on the upstream and downstream sides of the foundation excavation because of stress relief should be removed or rock bolted for safety.

We agree with the first US team that the in situ stresses in the rock should be disregarded in analyses of stress and deformation of the dam foundation.

It is quite important to place a grout curtain for seepage cutoff, as is now planned, under strict control of grouting pressures. Although all suspect rock exposed in the foundation excavation should be removed, a good consolidation grouting program under low pressures should be applied to fill undetected cracks resulting from stress relief. It is also important to have a deep, comprehensive drainage system in the foundation and abutments downstream of the grout curtain, and a monitoring system to be certain that adequate drainage is occurring.

For analyzing the dam foundation for sliding blocks downstream, it is the team's belief that the rigid block method of analysis is most applicable. If calculations indicate the possiblity of tension on any plane of rock, it is recommended that preventive measures be applied, probably post-tensioned steel bars or cables. Grouting, either cement or chemical, will not be effective in improving the characteristics of the rock because the joints are so tight. This comment also applies to the dam foundation.

The first US team reported some of the difficulties of reliably evaluating shear test results. While the total strength of a rock specimen during a test, either field or laboratory, may be representative, the separate contribution of friction and cohesion during a test is unknown. Tests on several specimens to separate these parameters are subject to errors caused by unavoidable differences in specimens.

After reviewing the data presented in the first US team report, as well as new data available since that report, we conclude that the criteria presented in that report are reasonable. This includes a recommended minimum factor of safety for sliding blocks equal to 3, and desirable factor of safety equal to 4.

For the shearing strength of the sliding blocks, we recommend the tan ϕ values suggested by the first US team, with cohesion as indicated by the more recent field tests.

The cohesion for each shear test specimen can be estimated from the original test data (not furnished the team) by applying an assumed tan Ø line to the Mohr's circle for each test specimen. The cohesion for the specimen would be the intercept of the tan Ø line with the zero normal pressure line. It is suggested that the "low mean value" of these intercepts be the selected design value before application of the factor of safety.

From the information at hand, it would appear that the friction coefficient for the intact rock part of the sliding blocks of syenite should be approximately tan Ø = 1.4, cohesion = 210 kg/cm^2 and for basalt tan Ø = 1.1, cohesion = 280 kg/cm^2. It is suggested that the friction and cohesion values for the portion of the sliding block along joints be as follows:

Syenite	tan Ø = 0.65	cohesion = 34 kg/cm^2
Basalt	tan Ø = 0.65	cohesion = 30 kg/cm^2
Uralitized basalt	tan Ø = 0.50	cohesion = 15 kg/cm^2

A combination of the "intact" and "joint" values should be applied in the analysis of abutment stability, depending on the continuity index.

It is important to have knowledgeable, experienced engineers and geologists examine and approve all exposed foundation rock before placing concrete on it.

IV. Underground Chambers

The powerplant, transformer, and surge tank chambers are located in the left abutment, mostly in intact syenite rock, but partly in basalt. These chambers are very large, and are likely to be very difficult to excavate and stabilize. This is not only because of their size and the jointed nature of the rock, but also because of the high in situ stresses in the rock. The finite element method of analysis, which is probably the best method available for this purpose, predicts that there will be several centimeters of rock deflection into the chambers. In practice, it is likely to be much more than this and will consist of block movement as well as elastic deformation. The amounts and locations of movements are largely unpredictable and must be recognized and dealt with during excavation. It is the team's opinion that the excavations should be done in small increments, with rock bolting and shotcreting as temporary minimun support.

A problem recognized by the team is the location of the three large chambers near each other. This is highly undesirable, because the relaxation zone of rock around one chamber will overlap with the rock relaxation zone of the adjacent chamber. It has been suggested that the transformer chamber be moved above ground for other reasons than rock conditions. This would solve the problem of overlapping zones of relaxation, and from the standpoint of possible difficulties underground, is strongly recommended. At the very least, there should be a minimum of two chamber widths of intact rock between chambers, and preferably three widths.

It is suggested that a similar criterion be applied, where possible, to the design of tunnels and other adjacent underground openings. Where this is not practical, special excavation and stabilizing procedures will probably be indicated.

VI. Jin Long Mountain

Landslides have occurred on Jin Long Mountain and underground conditions favor reactivation or initiation of major landslides into the reservoir as the water rises. From examination of rock conditions in two of the adits, it appears to the team that there is a high risk of an approximately 50-meter horizontal depth of decomposed rock sliding into the reservoir. This could endanger the diversion intake structures and create large waves.

It is unlikely that the potential slide material can be confidently stabilized by internal drainage or token toe support. Possibly the intake areas could be protected by rockfill buttresses during the diversion period. Possibly the best solution to this problem would be to excavate much of the material involved and place it in a toe berm or in the upstream cofferdam. The team believes solutions to the problem merit further consideration, particularly because stability analyses are not reliable for prediction of stability of such a mass of combined clay and decomposed rock. Such analyses, however, might be helpful in evaluation of possible remedial action.

By assuming that the potential slide mass is currently at the point of failure, factor of safety of 1, the destabilizing effect of raising the reservoir can be reasonably calculated. This will show a theoretical factor of safety of less than 1. The benefits of stabilizing berms, or unloading part of the mass at the top can then be calculated using the same shear strength assumptions originally assigned to reach factor of safety 1. The team believes the minimum acceptable factor of safety, including stabilizing berms and top unloading, should be 1.5.

The preceding would cover the condition of reservoir filling, whether in the construction or operation phase. A new analysis should be made in a similar manner to evaluate the effects of reservoir drawdown. However, such effects could be substantially reduced by appropriate installation of a drainage system.

The team also recognized another possible problem with respect to the stability of Jin Long Mountain. The two adits examined penetrated the weathered basalt, a considerable distance of intact basalt, and a thick layer of competent limestone. The adits ended in a layer of clay several tens of meters thick. It is understood that this clay, which is gray in color and quite firm, is the clay being mined for commercial purposes. There did not appear to be evidence of sliding movement in the clay, but the steep slope of the rock and clay beds toward the river is cause for concern.

It is the team's general opinion that there may be low risk of the large mass sliding into the reservoir, but the consequences of such a slide could be severe, even catastrophic. Thus, it is recommended that investigations be continued to evaluate the risk of such an event. Also, the team recommends wave studies to analyze the probable effect of such a slide in the unlikely event it would occur. A report by the USBR on model studies of landslide-generated waves for Morrow Point Dam was furnished to assist in this work.

STRUCTURAL ASPECTS

I. Observation of Structural Model Studies

A. March 23, 1983, Qinghua University, Beijing. - A structural model of the double curvature arch dam had been made to investigate the static stress distribution in the dam. A finite element analysis was also made at the university. The team was told that the results of the test and the analysis are close both in magnitude and in distribution. The three-dimensional finite element analysis included a portion of the dam foundation.

B. March 25, 1983, Tianjin University, Tianjin. - A dynamic structural model (scale 1:3000) of the arch dam was made to study its vibrational behavior with the reservoir empty and a rigid foundation. The vibrational modes of the model were studied using laser beams to produce holographic interferometry. The measured mode shapes and frequencies were first checked against analytical solution and then used as input to a stress analysis program to compute the dynamic stresses in the dam. For an earthquake of maximum ground acceleration of 0.2 G, the maximum dynamic stress was cited to be around 5 kg/cm^2. This number appears very low. After discussion with the investigators, it was found that the low stress was caused mainly by the following two assumptions:

1. A combined influence coefficient of 0.25 was used. This coefficient, in effect, cuts the earthquake load by a factor of 4.

2. The response spectrum used was such that the first mode fell in the descending portion of the curve with corresponding acceleration about 0.16 G.

C. March 28, 1983, Chengdu Institute of Science and Technology. - Two structural models were constructed (scale 1:400) for both static and dynamic tests. These models included foundation rock with different elastic properties as determined by the field investigations. For the static case, deflections and stresses were measured. In the dynamic test, natural frequencies and mode shapes were determined. These data were then used as input for a dynamic stress computation program. The reservoir was assumed to be empty. It was stated that the full reservoir case and dynamic stress measurement will be performed in the future.

II. Presentation by CHPSADI

The stress analysis of the dam was presented by Engineer Ji Jia Rong, Head, Structural Analysis Section, CHPSADI. This work was performed by Institute of Engineering Mechanics, Academia Sinica, Hydraulic Engineering Research Institute, Qinghua University, Tianjin University, East China Institute of Hydraulic Engineering, and CHPSADI. The average manpower involved at each organization was about 4 man-years.

Stress analysis of the dam was made for both the arch gravity dam and the double curvature arch dam. The analysis was made by the complete trial load method. Results of the trial load analysis were checked against the finite element analysis and the model tests. The finite element analysis programs cited were HXME (East China Institute of Hydraulic Engineering), Structural Analysis Program (SAP), Arch Dam Analysis Program (ADAP) (both University of California, Berkeley), and Automatic Dynamic Incremental Nonlinear Analysis (ADINA from MIT). Natural frequencies and mode shapes were also computed by the finite element method and checked against model test results.

Design criteria under normal loading was determined as follows:

Allowable Compression	80 kg/cm^2	
Allowable Tension	10 kg/cm^2	(upstream)
	15 kg/cm^2	(downstream)

Three different layouts of a double curvature arch dam were analyzed.

1. Foundation on the weak right abutment not treated. Maximum thickness of dam 75 m (original scheme).

2. Foundation on the weak right abutment consolidated by grouting. Maximum dam thickness 60 m (improved scheme).

3. Reduce foundation excavation by 10 to 60 m after treatment. Maximum thickness 65 m (reduced excavation scheme).

Maximum stresses under normal loading (waterload + silt load + temperature load) based on trial load analysis are:

	Scheme a.	Scheme b.	Scheme c.
Compression (kg/cm^2)	-77.5	-78.3	-77.6
Tension (kg/cm^2)	17.8	15.5	15+

Basically, all three schemes satisfy the design criteria stated above. The team was told that tension stress on the abutment of 40 kg/cm^2 was computed using the finite element analysis method.

For water load, the results of the trial load analysis, the finite element analysis, and the model tests were basically the same.

For earthquake analysis, both spectral analysis and the time-history analysis were performed. The maximum acceleration was 0.20 G. The accelerations in three directions were H:H:V = 1:1:0.5. The first ten modes were included in the analysis. Results of both analyses are comparable.

III. Summary of CHPSADI Presentations

o The geometry of the arch dam was determined by the trial load analysis. Design criteria are basically satisfied. The results are considered reliable because different methods gave similar results.

o The determination modulus of the rock foundation is important to the stress distribution in the dam.

- o The dam can be constructed without special foundation treatment.

- o Considering the cost, the stress distribution, and the earthquake resistance, the double curvature arch dam is recommended.

IV. Group Discussions

A list of 24 questions on seven subjects was given to the team before leaving the United States. The following discussions are on subjects II and IV; i.e.:

II. Static Analysis and Problems about Ertan Arch Dam
IV. Dynamic Analysis of the Arch Dam and Its Problem

Discussion started in the afternoon of April 1, 1983. The structural group included 17 Chinese engineers/researchers, representing eight institutes or universities. The discussion lasted until April 4, 1983. The content of the discussion was the same as the list given to the team; however, the sequence varied. A summary of the discussion is given in the following.

1. Trial load analysis is still being used as the main tool for the design of arch dams in the United States. This is a proven method with a set of generally accepted criteria to evaluate its results. Many arch dams have been designed and constructed successfully using trial load analysis. Many organizations have programed the trial load method to make it a handy and powerful tool for the dam designers.

2. In the last 10 years, more and more hydraulic structures were analyzed by the finite element method. It, too, is a powerful tool for stress analysis; however, the finite element method has not replaced the trial load method as the main design tool for arch dam design. This is mainly because there are no generally accepted criteria to evaluate its results, notably the stress concentration near the arch dam foundation caused by the discontinuity of the geometry of the dam. Most design offices used the finite element method as a supplementary analysis for the trial load analysis to perform the dynamic analysis and to compute the stress distribution of special structures. The finite element method can be very useful in hydraulic structure design once its capability and limitations are fully understood by the engineer.

3. In the United States, the most commonly used finite element programs for the analysis of arch dams are SAP and ADAP, both from the University of California. While SAP is a general purpose structural analysis program and ADAP is a special purpose program for arch dam analysis, both programs are based on the same theory and assumptions; therefore, similar results can be expected.

4. For dynamic earthquake analysis of an arch dam, linear elastic analysis is still used in the US. There are several nonlinear analysis programs which in principle can perform a nonlinear earthquake analysis of an arch dam. To the team's knowledge, none has been attempted. This is because of the uncertainty in the nonlinear material properties to be used, especially the foundation rock, and because of the excessive computing cost involved. The best documented dynamic analysis of the arch dam is the design report of Auburn Dam by the USBR. The team brought a copy of this report and left it with CHPSADI.

5. With regard to dam-reservoir interaction of an arch dam, the current state-of-the-practice in the United States is to use the added mass method. The formulation is a modified Westergaard approach presented by Professors Clough and Zienkwicz in London, 1975, in their paper "Finite Element Method in Analysis and Design of Dams, Part C, Dynamic Analysis - Methods, Problems, Criteria." This approach assumes that the water is incompressible. This problem is presently being studied by a joint US-China research project involving University of California and the Water Conservancy and Hydroelectric Power Scientific Research Institute. This study may shed more light on the compressibility of the water. A copy of the Clough-Zienkwicz paper was left with the CHPSADI.

6. The dam-foundation interaction problem can be handled in one of the following two ways in finite element analysis:

a. Include a portion of the foundation rock in the finite element model.

b. Using special foundation element to simulate the elastic support. One example is the Vogt coefficient type of formulation. Copy of reference paper from January 1974 ASCE Structural Division Journal was given to CHPSADI.

7. Results of stress computations and structural model tests by various institutes and universities were presented. The computations were made by the trial load method and the finite element analysis. In the latter case, both the Chinese programs and the American program were used. Stresses from the mathematical models and the physical models are basically comparable with similar magnitude.

8. Arch Dam Stress Analysis System (ADSAS) is a trial load program of the USBR. The program uses the trial load formulation to form a flexibility matrix to calculate the load distribution directly without iterative adjustment. The program has dynamic capability but the dynamic version has not been released to the public. The USBR suggested that SAP or ADAP be used for the dynamic analysis of arch dams. This program was written for a Control Data Corporation computer. Extensive modification of the program may be necessary to run it on other computers.

9. According to engineers of CHPSADI, the stress level of the double curvature arch dam and the arch gravity dam are similar. Because the former scheme requires much less concrete, it would be a preferred scheme.

10. The quality control of the dam concrete is very important for the success of the project. In the US, a special concrete laboratory is set up at the construction site for any major project. This laboratory takes concrete samples and monitors properties on a continuous basis throughout the construction period. The test results are evaluated immediately on the site and are sent to the design office regularly for further evaluation.

11. Design based on very conservative rock moduli may not always be conservative. For example, a kink was shown on the arch center line. This was caused by the very low rock modulus estimated at that elevation. However, the rock modulus may very likely be much higher than the conservative

estimate; in which case, this kink will cause an unfavorable stress condition if the dam were actually built with such geometric configuration.

12. The finite element analysis of the powerplant was presented. It was pointed out that this study should be used qualitatively in decision making. The design of powerplant should not rely entirely on the finite element study.

13. The following studies should be considered in the design stage of the project.

a. Structural effects of the spillway piers on the arch dam should be investigated. The piers of the ski jump chute will influence the stress distribution in the arch. The massive piers of the surface spillway will cause additional load during earthquake.

b. The construction difficulties should be carefully investigated considering the large overhang of the arch dam and the many structures cantilevered from the surface of the dam. (The surface spillway has a downstream cantilever chute of over 20 m.)

c. The procedure for concrete cooling and the schedule for joint grouting should be developed. The stability of the arch dam during construction before joint grouting should be investigated.

d. A quality control program should be established to ensure the quality of the very large volume of concrete. The USBR "Concrete Manual" may be a useful reference.

V. Conclusions (Structural)

1. Structural analysis of the Ertan Arch dam was made using both the mathematical models and physical models by various institutes and universities to study both its static and the dynamic behavior. The team members agreed that the amount of work is certainly more than adequate for the feasibility study. The methods used for the stress computation are similar to those presently used in the US. In fact, three of the computer programs cited, SAP, ADAP, and ADINA, are American programs currently used by design offices in the United States.

2. Results of various stress computations and model tests by different organizations are comparable. This fact should establish the credibility of these results.

3. It is suggested that the preceding results be checked against the design standard in China to establish the structural adequacy of the Ertan arch dam. Any fine tuning of the geometry and additional research work can be performed in the design stage.

4. Because the stress levels of the double curvature arch dam and the arch gravity dam are similar, and the former scheme requires much less concrete, it is the preferred scheme.

5. Some additional studies are recommended in the design stage of the project. These recommendations are listed at the end of the previous section, Group Discussions.

HYDRAULIC MODELS AND HIGH VELOCITY FLOW

I. Observation of Hydraulic Model Studies

A. March 23, 1983, Beijing. - The team visited the Hydraulic Laboratory of the Scientific Research Institute of Water and Hydroelectric Power in Beijing. At the institute, studies have been conducted on a 1:120 scale model of the gravity arch dam scheme.

The scheme includes the following:

1. Underground Powerplant - Six 500-MW units

2. Center spillway with five gates (16 m wide by 17 m high) - 12,500 m^3/s.

3. Bottom outlets with four gates (5 x 6 m) - 5,000 m^3/s.

4. Tunnel spillway (13-m diameter) (1 gate-11 x 10 m) - 3,000 m^3/s.

In this scheme, the spillway buckets discharge at various elevations:

Chutes 1 and 5 are high approximately	1,130 m
Chute 3	1,122 m
Chutes 2 and 4 are low approximately	1,055 m

Top seal radial gates for the outlet works discharge under high head (100 m). The spillway flow from chutes 2 and 4 impinge against flow from chutes 1, 3, and 5. This impingement helps the energy dissipation by dispersing the jet before it strikes the plunge pool. Twelve series of model tests have been conducted at the institute on the gravity dam scheme. Apparently some dynamic pressure data was taken with strain gages.

B. March 24-25, 1983, Tianjin. - The team visited Tianjin University where five model schemes dealing with the double curvature arch dam were evaluated.

1. The first scheme (1:120 model scale) had the following:

   a. Center spillway with five gates (16 m wide by 6 m high) - 5,000 m^3/s.

   b. Midlevel outlets with six gates (5 x 6 m) - 6,800 m^3/s

   c. Two ski jump chute spillways with two gates each (12 x 16 m) - 6,000 m^3/s.

   d. Tunnel spillway (13 x 13 m) - 2,000 m^3/s.

The midlevel outlets are designed to impinge against the center spillway flow. One modification included six center spillway gates and seven midlevel outlets. This resulted in jet impingement on the right downstream bank.

2. The second model was built to study diversion through the first stage completion with the top of the dam at elevation 1145 m. It will be designed to pass a 100-year flood ($16,000\ m^3/s$).

This 1:120 scale model had six midlevel gates (5 x 6 m), three low level gates on each side which discharge through temporary conduits into the bucket of the two ski jump chute spillways, and two low level gates (5 x 6 m) which will be plugged in the future.

3. The third model was a 1:70 scale model of two of the center spillway gates and chutes and one of the midlevel outlets. The midlevel outlet had a 20° upward slope at the exit which impinged against the spillway flow. There was obvious energy dissipation in the air and dispersion of the jets before striking the plunge pool. Velocity data using a pitot tube and pressure data using water columns were collected.

4. The team returned to the first model which had been modified to show a design which was apparently suggested by the first US team in 1981. The scheme (1:120 model scale) had the following:

a. Center spillway with five gates (16 m wide by 17 m high) - $12,000\ m^3/s$.

b. Midlevel outlets with six gates (5 x 6 m) - $6,800\ m^3/s$.

c. Powerplant to make up rest of flow (approximately $1,500\ m^3/s$).

5. The morning of March 25, 1983, the team saw a 1:70 scale model of three draft tubes, a surge tank, and an 18-m-wide, 25-m-high tailrace tunnel. The model was operated to show both free flow and pressure flow in the tunnel.

While at Tianjin, the team also had the opportunity to view two video tapes of other model studies dealing with the Ertan Project. They included studies of an earlier alternative to place the powerplant under the central spillway of a gravity arch dam and the 1:70 scale model tests to study dissipation of energy by water nappe impact.

C. March 28, 1983, Chengdu. - The team visited the Scientific Research Institute of the Chengdu Hydroelectric Power Design Institute. They have studied two schemes for the double curvature arch dam on a 1:120 scale model.

1. The first scheme had the following:

a. Center spillway with three gates (16 x 13 m) - $4,800\ m^3/s$

b. Midlevel outlets with four gates (5 x 6 m) - $4,700\ m^3/s$.

c. Two ski jump chute spillways with three gates each (16 x 13 m) - $9,600\ m^3/s$.

d. Tunnel spillway - $2,000\ m^3/s$

2. The second scheme had the following:

a. Center spillway with five gates (16 x 13 m) - 7,000 m^3/s.

b. Midlevel outlets with six gates (5 x 6 m) - 6,800 m^3/s.

c. Two ski jump chute spillways with two gates (16 x 13 m) - 6,400 m^3/s.

d. Tunnel spillway - 2,000 m^3/s.

The center spillway crest elevation for these schemes is 1187 m. The ski jump crests elevation is 1189 m, and the midlevel outlets are at elevation 1100 m. There will be two low level openings at elevation 1060 m for temporary diversion once the dam height reaches 1145 m. The dam will also have two sets of three gates (7 x 8 m) each at elevation 1080 m which will discharge into the ski jump flip buckets for temporary diversion until the dam height reaches 1205 m.

The Institute has also studied the diversion scheme and problems associated with transporting logs during diversion. The concerns of energy dissipation were also studied. The team viewed a 1:50 scale model of the proposed diversion plan. The model was a large outdoor model of the 60-meter-high upstream cofferdam, the 18-meter-diameter right abutment tunnel, the 18- by 25-m left abutment tunnel, the 30-m-high downstream cofferdam, and the downstream river channel. The team observed logs passing through the left abutment tunnel and watched as the reservoir elevation increased to the point where the diversion tunnels changed from free flow to pressure flow. There was mention of a negative 4-m pressure in the crown of the tunnel during the transient condition from free flow to pressure flow.

D. March 28, 1983 Chengdu. - In the afternoon, the team visited the laboratory of the Chengdu University of Science and Technology. They have built and tested a 1:120 scale model of a hydraulic jump stilling basin for a gravity arch dam. The scheme consists of five center gates (16 x 15 m) and four midlevel outlets (5 x 6 m) which discharge into the spillway stilling basin. The stilling basin converges in the downstream direction to prevent secondary currents.

II. High Velocity Flow and Hydraulic Model Tests

The CHPSADI general presentation included:

A. Function of the Project

The main purpose of the dam is to provide electric power generation. The installed capacity will be 3,000 MW with a firm capacity of 1,002 MW (two units operating) and the remainder for peak use. The passage of logs will be a major consideration of the project. There will be no navigation due to the steep river gradient. There is no irrigation planned for the project.

The canyon is made up of syenite and basalt. It is 100 m wide at the dry season water surface elevation and 700 m wide at elevation 1200 m (future maximum water surface elevation). The design flood (1,000-year) is 20,600 m^3/s; the PMF flood (10,000-year) is 25,200 m^3/s; and the diversion flood (20-year) is 12,600 m^3/s. The riverbed immediately downstream from the dam axis (first 250 m) is underlain with syenite. Downstream from the syenite is a zone of second stratum basalt which has a very low erosion resistivity. Further downstream, where the diversion tunnels discharge into the river, the rock underlying the river alluvium is third stratum basalt having higher resistivity characteristics than the second stratum basalt.

B. Summary of Dam Types, Configurations Studied, and Recommended Scheme for the Feasibility Report

Several alternatives have been studied by CHPSADI. The gravity arch concept had two alternatives for the powerplant location. One alternative included two 600-MW units in an underground powerplant and three 600-MW units under the spillway. The other concept had all five 600-MW units underground. There were also two concepts for waterways to pass the flood. The team observed the models for both concepts. The first scheme used impacting spillway jets to break up the energy of the flow before striking the plunge pool. The second scheme made use of a hydraulic jump stilling basin. These are described under Observations of Hydraulic Model Studies, sections I.A. and I.D., both in this chapter.

Two thicknesses of double curvature arch dams have also been investigated including several alternatives to pass the design flood. The double curvature arch dam schemes all include a six-unit (500-MW) underground powerplant with two tailrace tunnels.

The underground powerplant has several advantages. The advantages center on the problem of the narrow canyon at Ertan and the difficulty of discharging the flood. The underground powerplant permits full use of the canyon to discharge the flood without major impact on the operation of the powerplant. Another advantage is the ability to work on two fronts during construction; the dam and powerplant construction will not interfere with each other.

Due to the thickness of gravity arch dams, flood flows would be concentrated further downstream on the second stratum of basalt. The low resistivity of this material to scour would result in serious erosion of the material in the riverbed and on both banks.

Because of these considerations, the double curvature arch dam and underground powerplant have been adopted for the feasibility study of the Ertan Project. The gravity arch will be the alternate scheme.

C. Problems of Hydraulics and Hydraulic Models

The problems dealing with hydraulics and hydraulic models are the following:

1. How to evaluate the decision on reasonable alternatives for a high head dam in a narrow site? Is there any better arrangement?

2. How to evaluate the method of model tests of the various alternatives? Are the simulation techniques appropriate?

3. Comment on the use of nonscouring velocities as a model technique for mobil beds.

4. Comment on plunge pool design and alternatives.

5. Will vibration caused by jet impact in the plunge pool affect the dam safety?

6. Comment on unsteady flow alternating between pressure and free surface flow in the tailrace tunnels.

7. Discuss surface cavitation resulting from high velocity flow and methods to prevent damage caused by cavitation.

III. Questions

A. Modeling Scour in Plunge Pools

Moveable bed models of both the gravity arch and double curvature arch dams were used to investigate the location and depth of the scour hole. Laboratory models in Beijing, Tianjin, and Chengdu have been built at 1:120 scale. In a 1981 meeting in Tianjin, a decision was made to use the same size bed material for all the various 1:120 scale model studies. The d_{50} size chosen was 30 mm for the syenite and 7 mm for the second stratum basalt. Tests were conducted for 3 hours on the model representing 33 hours on the prototype. We were told that the 7-mm material was also tested in the syenite areas at Chengdu and there was no significant difference in the scour hole depth. The 1:70 scale sectional model at Tianjin used 40-mm material. These sizes of material were selected on the bases of nonscouring velocities of 10 to 11 m/s for syenite and 5 m/s for basalt. The anticipated block size on the prototype is 0.5 x 0.5 x 1.0 m for syenite and 0.10 x 0.15 x 0.10 m for the basalt. These sizes are based on joint analysis of the foundation rock downstream from the dam.

The USBR practice is to use granular material sized using the length ratio (based on the Froude Law) or fall velocity scaling to represent loose riverbed material. Cement-sand mortar capable of being eroded by flowing water has also been used. The dimensions of the model scour hole are then used in a qualitative manner to compare various alternatives.

With a determination of block fracturing in sizes of 0.5 x 0.5 x 1.0 m and 0.10 x 0.15 x 0.10 m, d_{50} material of 8 and 1.25 mm, respectively, would be tested in a USBR hydraulic model of 1:120 scale. Analysis using fall velocity and shear stress techniques would also be conducted to size the model material.(1)

(1) "Sediment Engineering" - ASCE Manual and Reports on Engineering Practice, No. 54, 1975.

It is beyond the state-of-the-art to model scour depths into the underlying bedrock of river channels. One only has to look at the model studies conducted for Kariba Dam[2] to realize the difficulty of such a task. Model studies performed at SOGREAH Institute in Grenoble, France, predicted a scour depth of approximately 26 meters below the dam. After 10 years, the actual scour depth was 50 meters below the dam. There are some research efforts underway in the US to simulate the bedrock properties in a physical model, but to date, they have not had success.

It would appear that a parametric study of a variety of prototype scour holes would be a valuable check on physical model studies. Such studies have been conducted by Damle[3], Wu[4], Taraimovich[5], and Martins[6].

Pressure measurements have also been taken on most of the models of alternative designs. With the plunge pool invert at elevation 970 m, the highest pressure reading using a water column was 13 m above the tailwater for the feasibility study discharging 19,070 m^3/s. This feasibility configuration resulted in two scour holes when granular material was placed in the model. The center spillway middle level outlet scour hole (12,400 m^3/s) was located 228 m from the dam at a depth of 963 m and the ski jump scour hole (2,500 m^3/s each) was located 430 m from the dam at elevation 966 m.

Solving the equation for a submerged jet, the velocity at elevation 970 m would be approximately 17 m/s for the five center spillways discharging 6,070 m^3/s into a pool depth of 65 m. It is difficult to analyze the effect of the midlevel jets impacting against the spillway jet, but observing the model indicates the width of the jet has more than doubled and the impact velocity of the combined jet at the water surface is lower than the center spillway operating alone. The 13-m water pressure measured in the model fairly well represents the calculated velocity of 17 m/s. It appears that the concept of jet impingement used in the feasibility design reduces the jet intensity on the floor of the plunge pool.

The model studies for Ertan Power Project have encompassed a variety of design alternatives. Their results have been used to qualitatively compare various schemes. Pressure measurements have also been used to determine energy levels on the floor of the lined plunge pool. Once a final design concept has been selected, pressure transducers should be used to measure the dynamic forces on the concrete liner. The liner can then be designed to withstand these forces.

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(2) Hausler, E. "The Scour Hole Below Kariba Dam," USBR Translation No. 896, July 1973.
(3) Damle, P.M., Venkatramad, C.P., and Desai, S.C. "Evaluation of Scour Below Ski Jump Buckets of Spillways, Central Water and Power Research Station," Poona, India.
(4) Wu, Chain Min. "Scour at Downstream End of Dams in Tawain," JAHR, January 1973, Bangkok Thailand.
(5) Taraimovich, I.I. "Deformation of Channels Below High Head Spillways on Rock Foundations," Hydrotechnical Construction, No. 9, September 1978.
(6) Martins, R.B.F. "Scouring of Rocky Riverbeds by Free-Jet Spillways," Water Power and Dam Construction, April 1975.

B. Alternatives to the Feasibility Design

It would appear that the process used by CHPSADI to evaluate reasonable alternatives for a high head dam in a narrow canyon are similar to those used in the US, at least from a technical point of view. In the US, careful consideration would be given to the costs of the various alternatives. There would also be additional considerations based on the specific project. These would include environmental effects and, for Ertan, the capability to pass floating logs over the dam.

In the hydraulic group discussions, flip bucket designs for Amaluza Dam (Equador) and Auburn Dam (US) were discussed. The Amaluza spillway is composed of three chutes and buckets which discharge the flood flow longitudinally down the narrow canyon. The Auburn spillway flip bucket has teeth at its exit to disperse the jet along the river, thus reducing the jet intensity on the downstream river channel.

The feasibility design proposes a double curvature arch dam with an underground powerplant. The flood waterways include the following:

1.	Center free full spillway* - five 16- x 11-m gates	-- 6,070 m^3/s
2.	Midlevel outlets - six 5- x 6-m gates	-- 6,370 m^3/s
3.	Spillway tunnel - right abutment	-- 2,000 m^3/s
4.	Ski jump chutes	-- 4,860 m^3/s
5.	Powerplant	-- 1,490 m^3/s
		20,790 m^3/s

The team believes this design is feasible from a technical point of view and could be built as presently planned. The following alternative is offered by the team because they believe it is more cost effective and would eliminate the discharge of flood flows by the ski jump spillways onto the weak stratum of basalt downstream from the center spillway plunge pool. It would have to be studied in detail to determine if it is a feasible alternative. The proposal is as follows:

1.	Center free fall spillway*-five 16- x 15-m gates	-- 9,230 m^3/s
2.	Midlevel outlets-six 5- x 6-m gates	-- 6,370 m^3/s
3.	Spillway tunnel-right abutment (use diversion tunnel)	-- 3,700 m^3/s
4.	Powerplant	-- 1,490 m^3/s
		20,790 m^3/s

*Comparison of various center spillways - Table 3

The advantages include:

1. Save construction of ski jump chutes. These will be very difficult to construct given the steep canyon which is prone to landslides.

2. Right diversion tunnel can be used as future spillway tunnel.

3. Reduce number of high pressure gates from eight to six.

4. Tunnel spillway would discharge on third stratum of basalt which is better than the second stratum and far from base of dam.

5. This scheme has flexibility of operation.

Notes:

1. The second stage diversion plan would have to be studied because this scheme eliminates the ski chutes.

2. Table 4 shows the comparison between Ertan tunnel spillway and the Glen Canyon, Hoover, and Yellowtail tunnel spillways.

3. The tunnel spillway will require an air slot placed upstream from the vertical bend. Glen Canyon Dam spillway tunnels operated for 6 weeks in June-July 1983 and sustained serious damage. This gives further evidence for the need to design adequate air vents to prevent cavitation damage.

4. The right diversion tunnels invert at the inlet and outlet could be a few meters higher and the slope could be reduced to reduce the tailwater submergence when used as a spillway. Because it is a closed conduit during high diversions, this should not affect its capacity when used as diversion tunnel at design capacity.

TABLE 3

SPILLWAY COMPARISONS

	Mossy Rock	Kariba	Morrow Point	Ertan Feasibility	Ertan Alternative
Dam Height (m)	184	128	142	245	245
Discharge Center Spillway (m^3/s)	7,800	8,400	1,133	6,000	9,200
Size of Openings (m)	12.9x15.2	9x9	4.6x4.6	16x11	16x15
Number of Openings	4	6	4	5	5
Unit Discharge into Plunge Pool ((m^3/s)/m)	150	156	62	76	115
Head during Discharge Flood (m)	84	85	117	165	165
Approx. Thickness of Water Cushion (m)	78	90*	18.6	65	65
Velocity into Plunge Pool, Vo (m/s)	35.7	36	42.2	50	50
Thickness of Nappe into Plunge Pool, d (m)	4.2	4.33	1.46	1.52	2.30
Ratio, t/d	18.6	20.8	12.7	42.8	28.3
Velocity of jet-flow at base of plunge pool (m/s)	18.2	17.4	26.0	16.8	20.7

*Thickness of water cushion after 10 years of operation where the jet excavated the plunge pool.

TABLE 4
TUNNEL SPILLWAYS

	Glen Canyon Dam	Hoover	Ertan	Yellowtail
Dam Height (m)	216	221	245	158
Discharge through Tunnel Spillway (m^3/s)	3,900	5,665	3,700	2,605
Tunnel Diameter (m)	12.5	15.3	13	9.8
Control Gates at Surface (m)	12x16 (2)	Side Channel (Drum Gates)	16x11 (2)	7.6x19.6 (2)
Effective Head (m)	161.8	180	165	148

Note: An air slot has been designed and tested for Yellowtail. The tests verified that the air slot prevented cavitation for the test flow. Air slots are presently under study for Glen Canyon Dam.

C. Effect of Free Overflow Nappe on Vibration of the Dam

It is the team's opinion that the random vibration of the jet impact on the basin floor will not interact with the natural frequency of the arch dam. We are not aware of any such interaction between stilling basin and dam.

D. High Velocity Flow and Surface Cavitation

Cavitation occurs in a hydraulic structure when the pressure at a point in the water is reduced to vapor pressure. It occurs in flow zones where subatmospheric pressure results from sudden offsets or other surface irregularities. As the vaporous cavities are transported into higher pressure zones, they implode sending out high energy sound pressure waves. If the implosions occur close to a surface boundary, serious damage to the structure could result. The degree of damage can be related to a cavitation index (σ) and the time of exposure. The cavitation index is defined as:

$$\sigma = (Po-Pv)/(\rho V^2/2)$$

Where: Po = reference pressure
Pv = vapor pressure
ρ = water density
V = reference velocity

If $\sigma > 1.7$, no surface protection is required. If $1.7>\sigma>0.3$, surface roughness should be smoothed to a chamfer (R/H) described by Falvey[7]. Once $\sigma<0.3$, surface chamfers have to exceed 15 which is very difficult to ensure in the construction of a large dam. Aeration grooves should be considered once $\sigma<0.3$.

The proper introduction of air to the flow surface where cavitation is occurring reduces the effect of the high energy sound waves by considerably reducing the wave velocity. With the addition of 0.1 percent air by volume of water, the sonic velocity can be reduced from 1,000 m/s to approximately 300 m/s.

In the construction of a large dam, care in selecting quality materials and ensuring high quality workmanship are very important. This, however, is essential in waterways exposed to high velocity flow ($V \geq 15$ m/s). Flow surfaces should be smooth with no offsets into the flow or away from the flow and the flow surface profile should match the design shape.

Several examples of damage to USBR hydraulic structures caused by cavitation were discussed. They included a range of waterways from Yellowtail tunnel spillway to Stampede Dam high pressure outlets and the intake structure of Auburn diversion tunnel. It is essential that good design and construction practice be used on the Ertan Power Project to limit damage caused by cavitation given the height of the dam.

(7) Falvey, H.T. "Predicting Cavitation in Tunnel Spillways," Water Power, 1982.

With respect to USBR laboratory measurement techniques for cavitation studies, the following information was supplied:

1. In order to measure the acoustic wave lengths on the order of 20 mm, high frequency (20 - 50 kHz) instrumentation is required.

2. It is important that the transducer be placed close to the suspected area of cavitation.

3. The spectrum and change in spectrum are more important in the analysis than the sound pressure intensity.

4. Although air content should be measured, the USBR laboratory has only measured oxygen content to date.

5. A low-pass analog filter (10 kHz) is also used to "clean" the signal.

E. Ertan Surge Tanks and Tailrace Tunnels

With regard to the question of unsteady flow operation, sometimes pressure - sometimes nonpressure, our experience indicates that one should either have pressure or nonpressure but not both.

For hydroelectric powerplants with long tailraces such as Ertan, there are two alternatives. One is to construct the tailrace at such an elevation so that it always operates as open conduit flow. The second is to construct the tailrace tunnel such that it always operates as a closed conduit.

Due to the narrow canyon and large flood discharges at Ertan, there is a considerable range of tailwater (23 m). To design the tailrace tunnel for open channel flow would result in the loss of a large amount of power over the life of the project. USBR practice would be to close the powerplant during large flood discharges, but it is the team's understanding that Ertan powerplant will operate for the design flood of 20,600 m^3/s.

To construct a closed conduit tailrace tunnel requires a detailed design of the tunnel and surge chambers. The surge chamber design must meet the surge control requirements as well as provide the required submergence for the turbines. To prevent negative pressures within the surge chamber, adequate air vents should be provided.

The corresponding tailwater elevation for the minimum discharge of 680 m^3/s is 1010.8 m (this will ensure full flow in the tailrace tunnel). It should be noted that to maintain this flow will require three turbine units, each operating at a design discharge of 248 m^3/s.

IV. Areas of Concern which Resulted from the Hydraulics Group Discussion

1. More investigation is needed to determine the reliability of high pressure top seal radial gates operating under 100 m of head. Radial gates for Tarbela, Dworshak, and Libby Dam should be investigated.

2. A scour hole may develop at the downstream end of the plunge pool due to the high velocity flow passing over the end sill onto the weak basalt. This may require additional model tests.

3. The CHPSADI engineers suggested the possibility of a permanent dike downstream of the plunge pool to increase the water cushion in the plunge pool. This would have to be carefully studied before accepting it in the final design.

4. Investigation of scour potential where diversion tunnels exit into river channel. Riverbed protection or a flip bucket may be required.

5. A Standard Operating Procedure should be developed to pass the flood flow at Ertan Dam. The sequence of which gates will be opened and in what order will have an impact, for instance, in the center spillway plunge pool, both from the development of large eddy currents and scour on the plunge pool floor.

6. It will be imperative to keep rocks out of the lined plunge pool. Side slope protection will be necessary. A study to determine if riverbed material can move upstream into the basin should be made and such movement prevented in the final design.

7. Current USBR policy requires low level outlets to be able to evacuate the reservoir in case of emergency. It is recommended that an emergency low level outlet be considered in the design for Ertan Dam.

V. Conclusions (Hydraulic)

1. The team had the opportunity to visit hydraulic model studies in Beijing, Tianjin, and Chengdu. There were approximately 10 models plus the videotapes of additional studies at the University of Tianjin and the Nanjing Hydraulic Research Institute. The number and type of hydraulic model studies indicates the importance CHPSADI has placed on selecting the proper design alternative. Most of the studies to date have been conducted on a 1:120 model scale. This is appropriate for feasibility level studies. Additional models of specific waterway passages will be required once the final design is chosen. These models would be on the order of 1:25 to 1:50 scale and would study items such as the detail of the spillway piers, pressure measurements of the middle level outlet chute, and other details where questions arise in the design process. If a tunnel spillway similar to the design of Glen Canyon were selected, model studies would be required to size and locate the air slot. Model tests to date have used movable bed material and pressure measurements in the plunge pool. The model tests support the plunge pool feasibility design for the central spillway. This design is further checked by comparing it with similar free fall center spillways such as Mossy Rock Dam, Kariba Dam, and Morrow Point Dam. The model study approach used by CHPSADI is similar to that used in the US for a feasibility level study.

2. The team recognizes the concern expressed by CHPSADI for the safe, reliable release of the flood discharge for Ertan Dam. The feasibility design of the waterways is technically feasible if given proper consideration in the final design. The team has offered an alternative which would reduce the major flood waterways from four to three by increasing the discharge over the central spillway and through the right abutment tunnel spillway. This would eliminate the need for the two ski jump chutes which will be difficult to construct and whose jets impact on poor quality basalt in the riverbed. If the ski jump chutes are used in the final design, more effort is necessary to determine if protection of the riverbed is required and how to accomplish it. Two types of flip bucket designs were also discussed if the ski jump chutes are used.

3. The question of the effect of the spillway jet impact in the plunge pool on vibration of the dam was not covered by the team. We are not aware of any such problem on existing center overflow spillways.

4. The subject of cavitation damage to hydraulic structures caused by high velocity flow was discussed in detail. Ertan Dam will have several waterways where damage could occur if not properly designed. The cavitation index was defined and types of protection suggested for various ranges of the index. It is suggested that aeration slots be designed for flow surfaces where the calculated index is less than 0.3. Depending on the quality of workmanship on the flow surfaces a higher criterion for aeration slots may be desired. Several examples of damage and corrective measures at USBR structures were given.

The USBR laboratory investigations of cavitation using a low ambient (vacuum) chamber were presented. Laboratory measurement techniques were briefly discussed.

5. With regard to the question of unsteady flow operation, alternating between pressure and nonpressure, the team feels that one or the other should be used but not both. From the standpoint of power production potential, it appears that a pressure system is best. This will require properly designed surge chambers to adequately protect the electromechanical equipment as well as the tailrace tunnels.

I. Description of Construction Project

At a lecture at Dukou Guesthouse on March 30, 1983, Mr. Xin Shi Zhi, Chief Engineer for Construction Division, gave a review of the following:

- Construction schedule and site facilities;
- River diversion and log drive;
- Excavation of the dam foundation;
- Concrete placement;
- Aggregate sources; and
- Transportation system

A. Construction Schedule

The overall schedule is 13 years long. A first stage power development of a partial group of machines are to be installed and operating in 10.5 years. The first 2 years of the schedule are for the construction of living quarters for single people, a 15,000-person townsite at Fangjiaou on the plateau above the existing access road, a rebar fabrication yard, truck repair shops, and especially for the difficult second access road on the right bank with a new bridge at Tongzilin connecting to the existing left bank asphalt paved access road which will be improved.

Years 3, 4, and 5 will be for excavation of the tunnels and the abutments of the dam. Abutment excavation is 50 meters deep in places and totals 4.5×10^6 m^3. The next 5.5 years (6 through 10.5) will be devoted to concrete placement in the dam up to elevation 1145 m with peak production of concrete at 6,000 cubic meters per day.

Construction will be carried out continuously on a rolling, three shifts per day basis. There will be a total of 25,000 men employed at peak of construction.

After reaching first stage power generation at elevation 1145 m, concreting of the dam will then continue at a slower rate of 80,000 m^3/mo until completion.

Powerplant excavation will take place during years 6, 7, and one-half of 8, followed by first-stage powerplant concrete in years 8, 9, 10, and one-half of 11, with second stage concrete and equipment installation through the last 4 years (10 through 13).

There is no problem with reservoir filling because of high annual flow.

B. River Diversion and Log Drive

Flood stages are forecast as follows:

5-year	Peak	9,400 m^3/s
20-year	Peak	12,600 m^3/s
50-year	Peak	14,600 m^3/s
1,000-year	Peak	20,600 m^3/s
PMF (10,000 year)	Peak	25,200 m^3/s

Diversion is designed for the 20-year flood of 12,600 m^3/s. Two alternatives are being considered: an overflow cofferdam and a nonoverflow system with a 60-m-high upstream cofferdam. The lower overflow cofferdam would not contain the 5-year flood of 9,400 m^3/s.

If the overflow scheme is used, flooding of the foundation worksite must be anticipated on an annual basis with work shut down for 3 to 4 months each year until elevation 1030 m is reached after 3 years when the 20-year flood would be contained.

The difficult part of the nonoverflow scheme is placing the 1.1×10^6 m^3 upstream cofferdam in one dry season of 9 months. However, factors influencing the Institute's leaning toward such a decision are:

1. The need for an annual pass-through of the existing 500,000 m^3 log drive from upstream which is practical with the nonoverflow system because of the enlarged diversion tunnel on the left bank;

2. Loss of construction time accumulating in a total delay of 1 year in the first-stage power generation;

3. The lack of a flood warning communication system and because of the flashy nature of the floods, the lack of ability to predict floods in time to safely withdraw men and equipment from the deep excavation; and

4. The lack of stability in the streambed silts underlying the cofferdam, but above streambed rock at approximately elevation 960 m, and their erodability during overflow stage.

The nonoverflow upstream cofferdam has an upstream face of asphaltic concrete and a concrete panel wall cutoff to bedrock. There are diversion tunnels on each side with the one on the left sized to pass the annual log drive.

C. Excavation of the Dam Foundation

Excavation will be accomplished in three stages: 1) above the crest, about 70 m; 2) between elevation 1030 m (present access roadway on left bank) and crest elevation 1205 m taking some 3 years; and then 3) below road elevation to bedrock at elevation 960 m. The latter will be accomplished in 15 months after closure of the river.

Abutment blasting will employ presplitting methods and mucking will be done from below with electric shovels and 40-ton trucks to haul to waste and to stockpiles for later crushing. (In view of the team "Ruptures" of the rock face from high in situ stress relief, if occurring, should be treated as a safety and scaling problem. The final stable face is to be cleaned with air and water jet before concreting.)

Concrete will be placed from a cross-river 20-ton cableway in either 6- or 8-m^3 buckets. There is 4.5×10^6 m^3 required between the lowest foundation at elevation 960 m and the first stage at elevation 1145 m.

There will be two batching plants, one at elevation 1205 m upstream and another at elevation 1100 m downstream. A post-cooling system for the concrete will be required prior to construction joint grouting and is presently under study by the Institute. Concrete placement has been simulated by computer for the 6-m^3 batches.

D. Aggregate Sources

Natural aggregates are available from the Anling watershed but these will require a rail and road haul of some 70 km total and would necessitate destruction of tillable land. Accordingly, present plans are to crush both coarse (up to 15 mm) aggregates and sand. Quarry rock will come from excavations and from limestone quarry pits upstream of the dam, 18×10^6 m^3 are required. Man-made sand is reported to require about 10 percent more cement to make a workable concrete mix. Economic studies underway at the University indicate the cost is equal except for the right-of-way problem.

E. Transportation

Consideration for transportation of various materials to the site include an 18-km extension of the railroad from the existing railhead at Tongzilin along the left bank, or to build a new road on the right bank with a bridge at Tongzilin.

The existing left bank road will be improved. Conveyors are planned for the crushed aggregate transport to the batch plant. Cement will come from existing plants in the Dukou region.

II. Site Visits

On Friday, March 31, a site visit was conducted by Mr. Xin Shi Xhi, Chief Construction Engineer, and his colleagues. Starting at the Dukou Guesthouse, the Jinshau River was crossed by ferry boat and the Anling River by bridge. A stop was made at Fangjiaou, future townsite, and the Tongzilin railhead, shop sites, and on the left abutment of the dam where a trip was made to the top of the Jin Long landslide area, presently the site of a commercial clay mining operation and a country hospital. An overview of the project site was had from the top of the slide at elevation 1500 m, as well as other levels on the way down through a series of about 15 switchbacks on this steep road. From the top Quarry No. 4 site was observed; later Quarries Nos. 1, 2, and 3 were observed from across the river well upstream of the damsite.

There was a lunch break at the workman's barracks installation at Santan Gulley.

Flow in the Yalong River was estimated at 600 m^3/s and had a color indicative of snow melt. This was indicated to be low flow.

After lunch the diversion intakes, cofferdams, and the powerplant adit were observed. It was noted that the ambient temperature in the adit is about 27 °C. A walk into the adit to the heading at about 500 meters was made and the various rock strata observed before returning to Dukou in the late afternoon. Enroute the Tongzilin damsite was observed in passing.

The next day, Seemel revisited the powerplant adit and adit No. 5 as well as adit No. 11 at elevation 1039 m on the right bank where various weaker rock strata and fault F-20 were observed. Of particular interest was the uralitized basalt bands intercepted by the adits and the many in situ rock tests.

III. Discussions at Dukou

Discussions between the nine members of the Institute's construction group and Marcotte and Seemel were held Saturday and Sunday, April 2 and 3. A short discussion between the construction group and Chaing took place on Friday, April 1.

Additional discussions about construction matters took place at general meetings on April 3 and 4, and in Chengdu a limited meeting was held April 5.

During the discussions, all of the questions in Section V of the Technical Interchange Problems (Appendix E), prepared by CHPSADI, were discussed and answered to the extent possible.

A. Diversion

The recurrence period of 20 years for the diversion flood was considered practical for construction planning on Ertan Dam construction.

The upstream cofferdam alternatives noted were considered. Although it was not clear in the discussion why a clay core dam could not be built of clay from the nearby Jin Long clay mine, the proposed panel-wall cut-off is considered feasible. It was noted that equipment used in America for concrete panel-wall construction is similar to the Chinese equipment. As an example of the practicality, the double wall cutoff at Manicougan No. 3 Dam in Quebec, Canada, is over 100 m in depth. It is not considered necessary to place reinforcing steel in the proposed wall.

It will be necessary to stabilize the slope of the riverbed material between the downstream toe of the cofferdam and the permanent dam bedrock foundation level. A reversed filter with a heavy rock zone on the slope surface is suggested for erosion protection. The question of landslide risks for this slope was not addressed by the team, but should be studied.

Regarding the practicality of the two diversion plans, the nonoverflow cofferdam plan is obviously more conservative and an economic study was suggested to determine the comparative costs of the alternatives.

The general layout of the diversion tunnels is suitable except that the intakes could be blocked by a landslide from Jin Long Mountain. It is necessary to take precautionary measures in locating the intakes and in providing protection upstream.

The team urged that careful consideration be given to dual use diversion tunnels for spillways or for tailraces for economic reasons and because of problems from overlapping of relaxation zones in multiple parallel tunnels during construction.

B. Log Drive

In Chengdu, the alternative plans for the annual log drive down the Yalong were discussed and sketches reviewed. While there is some familiarity with logging practices in North America among members of the team, it is recommended that the Institute retain an expert on log handling from a major American or Canadian lumber or paper company. Certainly the existing studies indicate careful consideration is being given to this operation. The forecast is for over 1.38×10^6 m^3 of wood annually to be passed around the dam in the future. The team did recommend that the final design provide flexibility; i.e., if the conveyer option is selected, design the tunnels so conversion could be made to water conveyance. Truck haul from a collection point in the reservoir is also a valid consideration.

C. Excavation of the Dam Foundation

A profile of the proposed excavation for the abutments of the dam was displayed and it was noted that although the quantity of excavation (4.5×10^6 m^3) as controlled by the weathered zone is much greater, the plan is similar to that used at Yellowtail Dam and at Glen Canyon Dam except the excavation will be deeper. Weathered rock 50 to 60 m thick will be removed. Demarcation of acceptable rock is apparent when walking through the exploratory adits.

Tracked drilling equipment was discussed and the team suggested that CHPSADI, working through the American Embassy Commerce Advisor, could obtain catalogs on various equipment and arrange for the purchase of whatever equipment they decided upon. This same holds true of obtaining information on US computers and programs for construction simulation and analysis. It was suggested that a general list of equipment types be prepared and presented for this purpose.

D. Cableways

Cableways were next considered with examples from Itaipu (Brazil), Yellowtail, and Glen Canyon Dams. The Itaipu cableway may make a good example, but the other two (about which literature is presently available) certainly are good examples. The Glen Canyon cableway at 50 tons capacity is said to have been the largest in the world.

It was emphasized that cableways in US practice are specially designed using standard components by specialists who could be contacted through the American Embassy as stated previously.

E. Underground Excavation

Excavation of the tunnels, powerplant cavern transformer vault, and surge chamber was also discussed. It was noted that these caverns will all have reinforced concrete arches because of rock conditions. At a general meeting on April 4, 1983, the team brought up the question of eliminating the transformer vault by moving the transformers and switchgear to the surface.

The stability of the rock mass between underground excavations is an unknown factor with no existing technology for absolute determination of stability before actual excavation. Therefore, it is best to avoid the problem by eliminating chambers which have viable alternatives such as placing the transformers above ground. It may be necessary, in any case, to heavily reinforce the tunnels and walls of the caverns with rock bolts, mesh, and shotcrete as the excavation proceeds. Caution will have to be exercised to avoid accidents from rock bursts resulting from the high in situ stresses. Systematic tightening of the rock bolts and sealing of loose rock will probably be necessary. This is recognized as an expensive procedure. Due to high in situ stresses, there are strong reasons for avoiding any other excavations underground with overlapping zones of rock relaxation such as for the transformer/switchgear vault.

The Institute Electrical Department cites an average length of 500 meters of bus duct to reach the surface directly. However, the generators are at approximately elevation 1000 m, and the average ground elevation above the machines is 1300 meters. It is also reported that the bus duct will require 1000 metric tons of aluminum. It is the team's belief that based upon relative cost, bus duct costs cannot outweigh the impact of problematic vault excavation. The LG-2 underground powerplant at James Bay, Quebec, owned by Hydro-Quebec, has bus duct directly to the surface, which is a distance of 170 meters. It has 16 machines rated at 333 MW each, totaling 5,328 MW, and is in operation.

It seems worthwhile for the CHPSADI to conduct a very thorough review of these matters assisted by a recognized authority with experience on similar installations.

The necessity of sequencing tunnelling and concreting of the six draft tubes is considered a normal construction procedure. The 18.5-meter-wide tunnels, which are only 9.5 m apart, must be excavated in such a manner as to never attempt to remove a greater width of rock than exists between two draft tubes at any given time; i.e., the excavation must be done in slices.

F. Concrete and Aggregate

Concrete design mixes, manufacture, and handling were reviewed. The cement is manufactured in the Dukou region and CHPSADI feels they will have good control over quality of the cement for the dam.

Aggregates, as noted previously, are available from several sources, but for various reasons, natural river aggregates have been ruled out. This is particularly unfortunate in the case of the sand because manufactured sand results in a harsh mix which leads to several undesirable side effects. The added cement required to bring the workability up to a tolerable level does not enhance the quality of the final product in any respect; in fact, it leads to truly lower workability regardless of slump, high water/cement ratio, and associated higher heat of hydration, therefore, requiring more expensive refrigeration for both precooling and postcooling of the concrete.

CHPSADI personnel expressed a concern for a means of reducing temperature of the mix to less than 10 °C and stated that the heat gain between batching and the forms of as much as 4 °C. This seems unusual because at Yellowtail, there was only about a 1.5 °C increase. Also, the 10 °C requirement was questioned because the Yellowtail requirement was for 15 °C. However, later it was noted that such requirements vary with ambient temperature and that Glen Canyon specifications required 10 °C. The team suggested that the Ertan designers establish a maximum temperature for placement which should not be unrealistically low due to the high cost of refrigeration. (It is noted both Yellowtail Dam and Glen Canyon used natural aggregates and low water/cement ratios.)

Handling methods of concrete were discussed and it was generally thought that concrete should be transported from the batch plant to the cableway lift point in buckets on a lowboy rather than using transit mix trucks. It was suggested that a construction simulation be run using 8-m^3 buckets instead of 6 because there may be considerable improvement in the placing rate which requires a peak of 6,000 m^3/d.

A question was raised about the practicality of using a downhole dump for quarry aggregates. Seemel suggested the methods used at Chicoasen Dam in Mexico be reviewed as they used a vertical shaft to move filter aggregates to the lower foundation level from the plateau of this record high dam. Dworshak Dam (US) had an underground crushing chamber connecting to the quarry floor with a 6-m-diameter, 165-m-long shaft.

Use of conveyer belts to move aggregate from borrow sites to batch plants seems appropriate.

G. Ski Jump Spillway

At the general meeting on April 4, the difficulty in building the ski jump spillway on each abutment was expressed. Concerns are that intakes cantilevering out from the face of the dam upstream 20 meters are difficult to construct, sidehill excavation would be difficult to hold, and the critical concrete surface finish to resist cavitation of lined waterways is extremely difficult to achieve. It is highly questionable if such waterways can be built with the required surface quality finish. Further, exposure to the weather will certainly result in deterioration over a period of years of the concrete surface.

H. The General Arrangement

The general arrangement of the construction facilities including the batch plants seems well thought out and the team has no further comment about such facilities except with regard to the access road network. Consideration should be given to a new high speed road on both abutments and to restricting the existing road to local traffic. This would be in the interest of safety as well as efficiency although the high cost of road construction along this steep valley is recognized. Such a road network could also be used for hauling logs from the reservoir, and thus, could eliminate part of the costly log handling facilities.

I. Reviews of Other Projects

Seemel reviewed a typical construction management job of similar magnitude as practiced in North America by talking about the organization and accomplishments of the Acres Canadian Bechtel, the construction management company at Churchill Falls, Labrador.

Marcotte described the Yellowtail double-curved arch dam project a multipurpose scheme which has proven very successful.

Two volumes of information on double-curved arch dams issued by the Bureau of Reclamation, both subtitled "Technical Record of Design and Construction," were given to the construction department of the Institute for future reference:

Yellowtail Dam and Powerplant
Denver, Colorado, May 1975.

Glen Canyon Dam and Powerplant
Denver, Colorado, May 1970.

VI. Conclusions (Construction Management)

1. The construction schedule as presented is difficult, but can be accomplished if construction planning of the present high quality is maintained. A 6,000-m^3/d peak for concrete placement will be an outstanding achievement.

2. The importance of quality in all phases of the work cannot be overemphasized. In the case of concrete, it is recommended that a search for natural aggregates be continued.

3. The first stage power option at elevation 1145 m is a viable consideration on a project of this magnitude.

4. The underground cavern excavation is critical and difficulties which are indeterminate by present state-of-the-art methods of testing may be encountered. It will be necessary to solve difficulties as construction progresses by practical methods.

5. It is recommended that experts with design and construction experience on large dams, tunnels, and underground powerhouses be retained for Ertan Dam.

6. The nonoverflow upstream cofferdam is preferred.

7. The freefall spillways as designed will be difficult to construct, but from a practical standpoint, the ski jump spillways will be extremely difficult.

8. Careful analysis must be made of the allowable concrete mass temperature so appropriate methods of cooling may be arranged.

9. Consideration should be given to developing a special cement of lower heat of hydration similar to Portland Cement Type II, for Ertan.

10. The ability to forecast any flooding during construction is desirable.

11. The annual log drive is important and can be provided for.

12. There is a need in studying construction alternatives to emphasize use of economic cost comparison.

APPENDIX A

LIST OF CHINESE PERSONNEL ATTENDING THE TECHNICAL EXCHANGE ACTIVITY

Name	Sex	Title of Profession	Major	Unit
Yin Kai-Zhong	Male	Deputy Chf. Engineer	Hydraulic Eng.	Chengdu Institute
Liu Ke-Yuan	Male	Deputy Chf. Engineer	Eng. Geology	Chengdu Institute
Wang Hun-Yuan	Male	Deputy Chf. Engineer	Hydraulic Eng.	Chengdu Institute
Li Yuan-Hui (Group Leader)	Male	Senior Engineer	Rock Mechanics	Chengdu Institute
Jiang Ge-Jie	Male	Senior Engineer	Eng. Geology	Hydroelectric Power Const. Admin.
Ke Zhi	Male	Asst. Researcher	Eng. Geology	Geology Inst. of Chinese Science Academy
Shao He-Gao	Male	Senior Engineer	Rock Mechanics	Chengdu Institute
Li Guang-Yu	Male	Asst. Researcher	Rock Mechanics	WuHan Rock & Earth Inst. of Chinese Science Academy
La Chen-Di	Male	Engineer	Rock Mechanics	Chengdu Institute
Shuen Zhong-Yuan	Male	Engineer	Rock Mechanics	Chengdu Institute
Zhang Chen-Juan	Female	Engineer	Rock Mechanics	Chengdu Institute
Hau Shi-Xin	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Li Wa-Zhan	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Lou Shu-Ying	Female	Engineer	Construction	Chengdu Institute
Wa Shi-Zhen	Male	Engineer	Eng. Geology	Chengdu Institute
Shao Zhong-Ping	Male	Engineer	Eng. Geology	Chengdu Institute
Zhang Qing-Ri	Male	Engineer	Eng. Geology	Army Corps 00300
Wang Yu-Tian	Male	Engineer	Eng. Geology	Army Corps 00300
Chen Chang-Ping	Male	Engineer	Eng. Geology	Army Corps 00300
Wu Ming-Gao	Male	Engineer	Hydraulic Eng. & Hydraulics	Chengdu Institute

Name	Sex	Title of Profession	Major	Unit
Liang Zhen-Xiang	Male	Asst. Professor	Hydraulics	Chengdu Science & Technique Inst.
Fang Tue	Male	Lecturer	Silt	Chengdu Science & Technique Inst.
Yao Zhong-Da	Male	Lecturer	Hydraulic Eng. & Hydraulics	Tainjin University
Min Shi-Wa	Male	Engineer	Hydraulic Eng. & Hydraulics	Hanjin Water Cons. & Hydro. Power Scientific Research Inst.
Wuang Zhong-Wa	Male	Engineer	Hydraulic Eng. & Hydraulics	Hanjin Water Cons. & Hydro. Power Scientific Research Inst.
Xing Xian-Lu	Male	Engineer	Hydraulic Eng. & Hydraulics	Chengdu Institute
Fu Pei-Fan	Male	Engineer	Hydraulic Eng. & Hydraulics	Chengdu Institute
Chen Zhi-Hun	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Li Yuan-Yuan	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Wang Zhao	Male	Engineer	Hydraulic Eng. & Hydraulics	Chengdu Institute
Yu Lu-Zhai	Male	Engineer	Eng. Geology	Army Corps 00300
Dong Bi-Zhang	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Shi Fang-Da	Male	Lecturer	Hydrostructure	East China College of Water Mgmt. Engineering
Yang Ro-Qin	Female	Lecturer	Structure	Qinghua University
Haang Zhi-Xian	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Lin Hun-Xing	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Chen Ai-Fan	Female	Engineer	Hydraulic Eng.	Chengdu Institute
Zheng Xiou-Ji	Male	Engineer	Hydraulic Eng.	Chengdu Institute

Name	Sex	Title of Profession	Major	Unit
Zhou Hua-Shong	Male	Engineer	Hydrostructure	Chengdu Institute
Pu Din-Sheng	Male	Engineer	Eng. Geology	Army Corps 00300
Lo Pei-Zhi	Male	Asst. Engineer	Eng. Geology	Army Corps 00300
Xin Shi-Zhi	Male	Chf. Eng. of Const.	Construction	Chengdu Institute
Lu Chung-Pu	Male	Asst. Professor	Construction	Qinghua University
Zhu Guan-Xi	Male	Asst. Professor	Construction	Tianjin University
Tan Wei-Tang	Male	Senior Engineer	Construction	Hydroelectric Power Const. Admin.
Huang Qian-Zhai	Male	Senior Engineer	Construction	Chengdu Institute
Lu Zhi-Chao	Male	Engineer	Construction	Chengdu Institute
Hu Pei-Ying	Female	Engineer	Construction	Chengdu Institute
Mu Jian-Zhi	Male	Engineer	Construction	Chengdu Institute
Xu Shou-Zhen	Male	Engineer	Eng. Geology	Army Corps 00300
Zhou Yuan-Biao	Male	Chf. Engineer of Hyd. Engineering	Hydraulic Eng.	Chengdu Institute
Chen Da-Sheng	Male	Deputy Researcher	Earthquake Eng.	Eng. Mechanics Institute of Chinese Academy of Science
Lo Xie-Hai	Male	Deputy Researcher	Dynamic Analysis of Dam	Eng. Mechanics Institute of Chinese Academy of Science
Cheng Hou-Qun	Male	Senior Engineer	Antiseismic Structure	Water Conservancy and Hydro. Power Scientific Research Inst.
Wuang Ki-Xian	Male	Lecturer	Antiseismic Structure	Tianjin University
Ji Jia-Rong	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Tan Zhong-Qian	Male	Engineer	Hydraulic Eng.	Chengdu Institute

Name	Sex	Title of Profession	Major	Unit
Mao Chou-Ping	Male	Engineer	Hydraulic Eng.	Chengdu Institute
Xin Ti-Qian	Male	Engineer	Interpretor	Chengdu Institute
Rin Zhi-Hui	Male	Engineer	Interpretor	Hydroelectric Power Const. Admin.
Li Chen-Chu	Male	Asst. Lecturer	Interpretor	Chengdu Geology College
Shou Qi-Qian	Male	Engineer	Interpretor	Chengdu Institute
He Qiao	Female		Interpretor	Chengdu Institute
Liang Huei	Female	Student	Interpretor	Tianjin Science & Technique Inst.
Wang Chou-Ying	Female	Student	Interpretor	Tianjin Science & Technique Inst.
Fong Huan	Male	Student	Interpretor	Tianjin Science & Technique Inst.
Yu Qin-Hai	Male	Student	Interpretor	Tianjin Science & Technique Inst.

APPENDIX B

UNITED STATES TEAM

FOR

ERTAN HYDROELECTRIC PROJECT

March 11, 1983

Joseph B. Marcotte, Jr.
Team Leader

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Dr. Tsung-Ting Chiang

Chief Hydraulic Engineer
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The process for selection of non-Federal members:
The National Council for US-China Trade, the Department of Commerce, and the American Council of Engineering Consultants solicited applicants and nominations from the private sector engineering community. Selection of non-Federal team members had a primary emphasis of providing specialists in the area of Geotechnology, Structural Design, and Construction Planning and Management.

APPENDIX C

ITINERARY

March 20, 1983	Team members Marcotte, Clevenger, Yeh, Burgi, and Seemel arrived in Tokyo.
March 21, 1983	Team member Chiang arrived in Tokyo. Entire team traveled via PanAm Flight 15 to Beijing. The team was met by Messrs. Zhou, Tan, Ren, and other members of the ministry staff as well as US Embassy staff Wardlaw and Robinson. The team was transported to the Yanjing Hotel. Mr. Wardlaw will travel with the team.
March 22, 1983	Toured "Forbidden City" in the morning. At 3:00 p.m., the team met with Mr. Zhao and key members of the ministry of Water and Power to review the itinerary for the visit and assure concurrence on the objective. Minor modifications in the itinerary were discussed. It was agreed that it would not be possible for the team to provide comprehensive answers to the many complex questions associated with the project. The team will discuss with their Chinese counterparts the various technical aspects and prepare a report summarizing the views of the team. Toured "Temple of Heaven."
March 23, 1983	Visited Qinghua University and Laboratories. University Vice-President Chang K.T. presented a description of the various aspects of the University including laboratory facilities. Staff members made a presentation of model studies and the finite element analysis conducted on Ertan Project. The team viewed the models which included a hydraulic model of one spillway alternative and a geologic model of the abutments. Visited the Water Conservancy and Hydroelectric Power Scientific Research Institute. Following a brief history and description of the institute by Sheng C.G., a presentation was made regarding hydraulic studies being conducted on Ertan Project. The team then had the opportunity to view the operation of a model of a second alternative spillway configuration.
March 24, 1983	Traveled to Tianjin by train. Housed in Tianjin University guest housing. Professor Dr. Shi Shao-Xi, President of Tianjin University, discussed the history of the various departments and specialties of the University. Staff members presented discussions on hydraulic studies being conducted by University laboratories. Several configurations of spillway discharges and a model of the tailrace tunnel were observed.
March 25, 1983	Observe structural model for dynamic studies using lasers. Traveled by train back to Beijing.
March 26, 1983	Toured Great Wall and Ming Tombs.
March 27, 1983	Traveled to Chengdu by CAAC Flight 4102.

March 28, 1983 Observed hydraulic model of a different spillway configuration and the cofferdams and tunnels and two structural models of the dam and a three-dimensional geologic model of the abutments. Left by train for Dukou at 7:00 p.m. Mr. Clevenger became ill and stayed behind. Mr. Wardlaw stayed with him.

March 29, 1983 Arrived Dukou 10:30 a.m. Brief discussion with Mr. Yin Kai Zhong on itinerary during stay in Dukou. Technical presentations were made.

1) General condition and function as well as study of dam configurations by Mr. Wang Hong Yan, Chief Design Engineer of Ertan Project.

2) Structural analysis by Ji Jia Rong.

March 30, 1983 Continued technical presentations.

(3) Stability analysis by Mr. Chen Yun Biau, Deputy Chief of Hydraulic Structures Division.

4) Construction management by Mr. Xiu Shi Zhi, Chief Engineer of Construction Planning Division.

5) Geologic conditions of the site by Mr. Chang Qingrui, Geologist.

6) Rock mechanics by Mr. Le Chendi, Engineer in rock mechanics test.

Mr. Clevenger departed Chengdu for Dukou.

March 31, 1983 Began visits to Ertan damsite and relevant area. Three groups were formed. The hydraulic and structural members inspected the damsite and several adits into the foundation. The construction management members reviewed the construction campsites, staging areas, potential borrow areas, and the damsite. One adit was inspected. The geology group waited in Dukou for Mr. Clevenger who arrived at 10:30 a.m. The geologic conditions and rock mechanics were reviewed with Mr. Clevenger in the afternoon.

April 1, 1983 Messrs. Clevenger, Seemel, and Wardlaw returned to the damsite. Technical discussion on hydraulics, structural, and construction management began.

April 2, 1983 Messrs. Clevenger and Wardlaw returned to the damsite. Technical discussions continued.

April 3, 1983 Technical discussion continued.

April 4, 1983 Technical discussions continued. Left Dukou by train for Chengdu at 4:00.

April 5, 1983	Arrived Chengdu 8:30 a.m.
April 6, 1983	Visited Dujiang Yan (2,200-year-old irrigation project).
April 7, 1983	Visited Dongfang generator factory in Deyiang. Mr. Chiang traveled to Beijing.
April 8, 1983	The team presented its findings to CHPSADI, Mr. Li Erding, Chief Engineer, Ministry of Water and Hydropower, and Mr. Pang Gia Zhenz, Chief Engineer, Hydroelectric Design and Planning, Water Resources Hydropower Institute.
April 9, 1983	Traveled to Beijing by CAAC Flt 4101.
April 10, 1983	Sightseeing and shopping.
April 11, 1983	Messrs. Burgi, Yeh, Cleavenger, Seemel, and Marcotte traveled to Tokyo by CAAC 929. Team separated and took various routes home.
April 12, 1983	Mr. Chiang traveled from Beijing to Boston.

APPENDIX D

PUBLICATIONS GIVEN TO CHENGDU INSTITUTE

1. Hydraulic Laboratory Techniques, USBR, 1980.

2. Technical Record of Design and Construction, Glen Canyon Dam and Powerplant.

3. Technical Record of Design and Construction, Yellowtail Dam and Powerplant.

4. GR-76-25 - Hydraulic Model Studies of Amaluza Dam Spillway.

5. GR-81-3 - Hydraulic Model Studies of Bartlett Dam.

6. GR-82-5 - Hydraulic Model Studies, Yellowtail Afterbay Dam Sluiceway.

7. Engineering Monograph No. 37, Hydraulic Model Studies for Morrow Point Dam.

8. Engineering Monograph No. 41, Air-Water Flow in Hydraulic Structures.

9. Engineering Monograph No. 36, Guide for Preliminary Design of Arch Dams.

10. Engineering Monograph No. 20, Selecting Hydraulic Reaction Turbines.

11. Engineering Monograph No. 19, Design Criteria for Concrete Arch and Gravity Dams.

12. Engineering Monograph No. 7, Friction Factor for Large Conduits Flowing Full.

13. Morrow Point Dam and Powerplant Foundation Investigations.

14. Morrow Point Underground Powerplant Rock Mechanics Investigation.

15. GR-75-9 - Concrete Performance in Yellowtail Dam, Montana, 10-Year Core Report.

16. GR-81-5 - Concrete Performance in Morrow Point Dam, 10-Year Core Report.

17. Design and Analysis of Auburn Dam, 5 Volumes (7 books).

18. REC-ERC-72-18 - Power System Stabilization with High Initial Response Excitation on Large Hydroelectric Generators.

19. Handout -

    - REC-OCE-69-5 - Experimental Study and Analysis of Draft Tube Surges.
    - REC-ERC-82-9 - Hydraulic Model Studies of Land Slides - Generated Water Ways - Morrow Point Dam, April 1982, C.A. Pugh
    - Finite Element Method in Anaysis and Design of Dams, Part C, Dynamic Analysis - Methods, Problems, Criteria - Clough and Zienkwicz, 1975.

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    c. C-1169 - Sliding Friction Tests of Bronze Alloys in Combination with Various Metals.

APPENDIX E*

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* THE TECHNICAL INTERCHANGE PROBLEMS *
*                                   *
*               FOR                 *
*                                   *
*    THE U.S.B.R. EXPERTS GROUP'S   *
*                                   *
*     SECOND INVESTIGATION TOUR     *
*                                   *
*             TO CHINA              *
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CONTENTS

PREPARED BY

CHENGDU HYDROELECTRIC POWER SURVEY

AND DESIGN INSTITUTE, MINISTRY OF

WATER CONSERVANCY AND ELECTRIC POWER

* This Appendix is a reproduction of the document furnished by CHPSADI

** After receipt of this document it was agreed between the PRC and US that expertise associated with turbine-generator sets, gates, and hoists would not be included on the US Team. Thus, answers were not provided for this set of questions.

I. High velocity flow and hydraulic model tests

The feasibility study of Ertan Hydropower Station recommends the alternative layout of double-curvature arch dam and underground powerplant. The flood discharge structures are mainly composed of overflow spillways and middle-level outlets. Besides them, there are a ski-jump chute spillway on either side of the central overflow spillway and a tunnel spillway in the right bank used as subsidiary spillways. All these spillways are with flip buckets. Such a flood discharge scheme is adopted mainly considering that the flood discharge of Ertan Project is very large, the head is high. The maximum power of flood discharge amounts to about 30000 MW, besides, the riverbed is rather narrow, while discharging check flood, the power of discharged flow dissipated in unit area of downstream riverbed amounts to 5 MW/M^2. The reservoir of Ertan Project is small compared with the flood volume so its ability of flood regulation is poor. That leads to discharging flood frequently and lasting a long time. Aiming at above-mentioned characteristics, the design principle of flood discharge and energy dissipation should be such that the discharged flow is separated as much as possible along longitudinal direction of the river, the nappe before plunging into water should be diffused and aerated sufficiently in order to reduce the flow power per unit area and the scour of the riverbed and two banks and to ensure the safety of the flood discharge structures and scouring protection works.

The expert group of U.S.B.R. suggested that the two ski-jump chute spillways and the tunnel spillway be canceled in order to reduce investment and engineering quantity of the project.

Both two above-mentioned alternatives have been studied and their hydraulic model tests have been made.

(The distribution of flood flow and scouring results of the two alternatives refer to the following table)

1. The preliminary analysis and comparison of the model test results

The model scales of the two alternatives are both 1:120. The model test with a mobile bed and that with a fixed bed have been both made. The model test with a mobile bed is simulated by granulay material considering rock masses downstream to the dam cut by joints. According to the geological investigation and comprehensive analysis, for syenite d_{50}=3 cm, non-scouring velocity=10-11 m/sec; for basalt d_{50} = 0.9 cm, non-scouring velocity = 5 m/sec. The results of the mobile bed model test are listed at following table. For the alternative No. 3 (recommended by the expert group of U.S.B.R.), under usual flood discharged only by overflow spillway, the scour of downstream riverbed will be the most serious. Its model test is continuing now at Tianjin University. The final report of the test has not been received yet. Refering to the measured results of the alternative No. 1 under maximum flood, the deepest point of the scour pit will be below the level of 958 m. From the viewpoint of scattering the flow and reducing scour downstream to the dam, the alternative No. 2 is more favourable than the alternative No. 3, but the investment and engineering quantity will be increased much.

2. Some views about the two flood discharge alternatives

The two alternatives have their own strong points and weak points. They should be further discussed and studied. There are some people who have following different views on the alternative No. 3:

(1). The alternative No. 3 adopts large discharge per unit width and its nappe concentrates at the dam toe. If the floor of the plunge pool breaks, it will influence the safety of the dam. Therefore, when the type of free overflow is adopted for arch dam in some countries the discharge per unit width is limited below 50 m^3/sec. M.

In order to prove the recommended alternative at feasibility study stage, we have studied the flood discharge design level of Mossyrock Dam of the U.S. (refer to the Brief Description of Ertan Hydroelectric Project). When the ski-jump chute spillway and the tunnel also discharge flood, the scouring ability of design flood (overflow scouring ability and energy dissipation ratio per unit volume water mass) is close to the level of that of Mossyrock Dam.

Therefore, it is not sufficiently reliable that whole flood is only discharged by the overflow spillway and the middle-level outlets if any other else flood discharge structure are not provided.

(2). Ertan Hydropower Station discharges flood frequently. The overflow spillway and the middle-level outlets both have large sizes. The latter bears high head so it is hard to avoid accident during operation. Therefore, the operation flexibility of the alternative No. 3 is quite poor.

(3). The free overflow nappe near the dam toe will cause vibration of dam. Whether the vibration can or not influence the safety of dam should be carefully studied.

(4). According to the model, the alternative is feasible too, but whether the model can or not well represent the actual condition of the project still isn't quite clear yet.

Generally, whether the alternative is or not reliable should be carefully studied.

The vibration of the dam caused by overflow nappe has been studied and measured.

According to the model test, the dynamic load caused by overflow nappe is random pulses. At the riverbed level 980 m, for various flood discharge combinations the superior frequency has been measured at the range from 0.2 Hz to 0.5 Hz. Comparing with the natural vibration frequency of the dam 1.69-1.73 Hz. (empty reservoir), it can't cause any resonance.

Problems:

1. Analysis and comparison about the reasonable alternatives of the double-curvature arch dam.

DISTRIBUTION OF FLOOD DISCHARGE AND

LAYOUT OF FLOOD DISCHARGE STRUCTURES AND ENERGY DISSIPATION TYPE	OPERATION CONDITION	DISCHARGE (M^3/SEC)	NO. OF OPENINGS	OVERFLOW SIZE OF OPENINGS (M)	SPILLWAY DISCHARGE (M^3/SEC)	DISCHARGE PER UNIT WIDTH AT BUCKET (M^3/SEC)	MIDDLE-LEVEL OUTLET NO. OF OPEN-INGS	MIDDLE-LEVEL OUTLET SIZE OF OPEN-INGS (M)	MIDDLE-LEVEL OUTLET DIS-CHARGE (M^3/SEC)
(1) OVERFLOW SPILLWAY	CHECK FLOOD	25200			15620	163			7700
AND MIDDLE-LEVEL OUTLETS	DESIGN FLOOD	20600	6	16 x 16	11880	124	7	5x6	7700
WATER CUSHION DOWN-STREAM DISSIPATION	MEASURED MAXIMUM FLOOD	12200			10672	111			
(2) OVERFLOW SPILLWAY,	CHECK FLOOD	25200			8217	103			6890
MIDDLE-LEVEL OUTLET, SKI-JUMP CHUTE,	DESIGN FLOOD	20600	5	16 x 11	6072	76	6	5x6	6814
SPILLWAYS AND TUNNEL SPILLWAY. DISSIPATION BY WATER NAPPE IMPACT.	USUAL FLOOD	7600			6070	76			
(3) OVERFLOW SPILLWAY	CHECK FLOOD	25200			16080	173			6890
AND MIDDLE-LEVEL OUTLET	DESIGN FLOOD	20600	5	16 x 17	12669	137	6	5x6	6814
DISSIPATION BY WATER NAPPE IMPACT	USUAL FLOOD	7600			MODEL TEST HAS NOT FINISHED YET				

R E S U L T S O F S C O U R D O W N S T R E A M

DISCHARGE PER UNIT WIDTH AT BUCKET (M^3/SEC)	NO. OF OPEN-INGS	SKI-JUMP SIZE OF OPEN-INGS (M)	CHUTE DIS-CHARGE (M^3/S)	SPILLWAY DIS-CHARGE PER UNIT WIDTH AT BUCKET (M^3/S)	TUNNEL SPILLWAY DISCHARGE (M^3/S)	SITUATION OF FLOW	ELEVATION OF THE DEEPEST POINT OF SCOURING PIT (M)	DISTANCE FROM THE DEEPEST POINT OF SCOURING PIT TO THE DAM TOE (M)	ELEVATION OF THE HIGHEST POINT OF THE HEAP (M)	DISTANCE FROM THE HIGHEST POINT OF THE HEAP TO THE DAM TOE (M)	REMARKS
220						THE NAPPES OF OVERFLOW SPILLWAY AND MIDDLE-LEVEL OUTLET DON'T IMPACT	948.4	181	1039.2	354	
220							947.7	169	1031.7	366	
							958.4	159	1019	270	
230			6382	100	7000	THE NAPPES OF OVERFLOW SPILLWAY AND MIDDLE-LEVEL OUTLET IMPACT	968	208	989	275	RECOMMENDED AT FEASIBILITY STUDY STAGE
227	4	16 x 11	4858	76			973	208	988	150	
							970	195	989.6	195	
						THE NAPPES OF OVERFLOW SPILLWAY AND MIDDLE-LEVEL OUTLET IMPACT	966.6	164	997.6	240	RECOMMENDED BY THE EXPERT GROUP OF U.S.B.R.
230							971.4	162	989.6	240	
227											

2. How to evaluate the method of model test with a mobile bed? How to estimate the difference between the results of model test and prototype conditions? How to improve the hydraulic model test?

3. How to adopt a reasonable measure to reduce the harm caused by flood discharge to an arch dam (specially free overflow through the crest).

4. Whether the unsteady flow alternating sometimes pressure sometimes non-pressure can be allowed in the diversion tunnel, rebuilt as a tailwater tunnel at the latter stage? What measures can be taken to eliminate such condition?

5. The principle of choosing the protection measure and the type of spillway against surface cavitation caused by high-velocity flow. About this problem the study situation and the operation experiences and the effects of finished projects in the U.S. The critical problem about the incipient cavitation number of indoor test. The problems on measurement and how to determine the sonic intensity for ultrasonic measurement. The problems about the influence of air content and the model law for cavitation tests. The mechanism of aeration to reduce cavitation and associate measurement techniques, instruments and equipments etc.

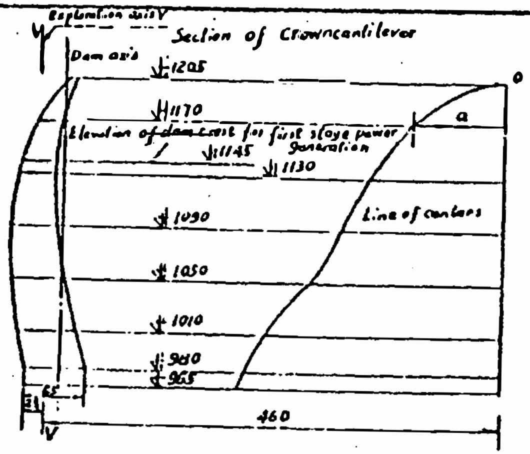
II. Static analysis and problems about Ertan arch dam

In selection of the configuration of the arch dam and analysis of its stresses, the program of complete trial load analysis worked out by our country is used. In the same time, we use the finite element method and the structure model test to check its result.

The foundation deformation property of the dam site is ununiform. The deformation modulus of the various rockmasses are very different. The modulus of deformation of the altered basalt zones is very low. It is about 1/10-1/25 of that of the syenite. It is necessary to determine an effective modulus of deformation which can represent the actual deformation property of the dam foundation when we use the trial load method to analyse stresses of the dam. We use the plane finite element method to calculate the effective modulus of deformation. i.e. at controling points in the foundation in different direction (arch direction and cantilever direction) under different loads (normal load, tangential load and moment) calculate the displacements of the foundation, using various moduli for different subregions of the foundation, then assume a homogeneous foundation which produces the same displacements under the same type of unit load. By this we find out the effective deformation modulus of the foundation.

The allowable stresses in the dam body is determined in accordance with the actual conditions of Ertan and the results computed by trial load method. The allowable stress is controlled by principal stress under the usual loading combinations the principal pressure stress should not exceed 80 kg/cm^2. The principal tensile stress upstream should not exceed 10 kg/cm^2. The principal tensile downstream should not exceed 15 kg/cm^2.

Section of Crowncantilever



Dimensions of Arches

Elevation of arch (m)	Extrados radius (m)	Arch thickness (m)	Central angle (°)		Arc length of Extrados (m)		Distance of Center a (m)	Effective deformation modulus of foundation (T/m^2)	
			α_r	α_l	S_r	S_l		right abutment	left abutment
1205	445	10	46.5	45	361.15	349.50	0	10×10^4	18×10^4
1170	384	24	52.5	51.5	351.86	345.16	90	10×10^4	18×10^4
1130	359.5	37	52.5	52	329.41	326.27	132.5	15×10^4	18×10^4
1090	335.5	47	48.75	49.5	285.46	289.85	164.5	10×10^4	18×10^4
1050	305	56	46.5	46.5	247.53	247.53	194.0	9×10^4	18×10^4
1010	254	62	40.5	30.5	179.54	135.21	240.0	15×10^4	18×10^4
980	224.5	65	26.5	17.5	103.83	68.57	260.5	21×10^4	21×10^4
965	217.5	65	0	0	0	0	267.5	21×10^4	21×10^4

Data used in the analysis:
nomal high water level upstream 1200m ; lowest tail water level 1011.8m ;
elevation of silt sedimentation 1065m; submerged unit weight of silt 0.9 T/m^3
internal friction angle of silt 0° ; Concrete unit weight 2.4 T/m^3 ; Linear thermal expansion coefficient 0.1×10^{-4} m/°C

The problems:

1. Please evaluate the results of stress analysis

The computer program we used is developed by our country. Although it has been used in a few projects, the time of its application is rather short. We hope to use the American program in common use, such as "ADSAS" to compute the stresses in the dam for the configuration we recommended in the feasibility report, so as to compare each other's results.

2. Please introduce the method to estimate the effective modulus of deformation of dam foundation and other methods to study the effects of soft rock foundation in the U.S.A.

3. What are the U.S.B.R. criteria of temperature control for concrete? It is said that U.S.B.R. stipulates arch dam design based on the "ADSAS" program and stipulates tensile stress not exceed 150 Psi under usual loading combinations, and tensile stress not exceed 225 Psi under unusual loading combinations. We don't know whether these requirements are suitable to the analysis of the finite element method and if these stresses contain the actions of uplift.

4. In computing temperature stresses, we use the experimental formula put forwards by USBR:

$$t=57.7/T + 2.44$$

How do you calculate the temperature stresses at further stage? Please introduce it.

5. How do you determine the size of the arch dam abutments with pads?

How do you study VOGT constant in analysing stresses in an arch dam with the trial load method?

6. What suggestions of improvement do you have for the dam configuration recommended at the stage of feasibility study?

7. Please introduce the "ADSAS" program or other methods and techniques about analysing arch dam stresses.

8. What technical measures should be adopted so as to guarantee the concrete strength in the high concrete arch dam?

III. Stability analysis of the dam abutments and its problems

There are no large discontinuities in the jointed rock masses in the dam abutments. We adopt the three-dimensional rigid block method and the method of plane finite element to study the stability of rock masses.

Slide block analysed is composed of a steep joint plane and a low angle joint plane. Under the loads and the combination conditions of the structural planes, tension will be produced on the steep joint plane of the block. Under the condition, we consider the rockmass can not bear the tension and the block slides along a direction on the low angle joint plane. Owing to neglect the shearing strength of the steep joint plane, the safety factors of stability of the rock abutments are relatively low. The safety factor of stability Kc 3 at the lower part of the dam (at its 1/3 height). The reasons are: 1. neglecting shearing strength on the steep joint plane. 2. that the shearing parameters used are on the low side.

Problems:

1. Using the rigid block method to analyse the stability of rock abutments in U.S.A. if there is tension on the steep structure plane. How do you deal with such a case in the analysis?

2. When determining the shearing strength parameters, do you use the average of peak values or the average of low peak value? How do you consider the safety factor in determining shearing strength used in the analysis?

3. How do you judge the stability of dam abutments from the results of rigid block method and of the method of plane finite element?

If an arch dam abutments are not stable enough, what method do you adopt as a basis for strengthening the abutments?

IV. Dynamic analyses of the arch dam and its problems

Ertan hydroelectric station is located in a seismic rigion. The dynamic analyses for the dam should be done in accordance with the proposals of Specialists Group of the Bureau of Reclamation (B.R.). This analysis has been carried out at the feasibility study stage in the following ways, Ertan Design Group had computed with a way called Imitative Static Method, which is in accordance with the Regulation--Aseismic Design Standard for Hydraulic Structure (SDJ 10-78), promulgated in China. This regulation is suited to the dams of height up to 150 meters. The regulation is based on dynamics theoretically, i.e., when the dynamic responses of different structure were analysed, a definite load distribution diagram was so drawn in the regulation that it not only represents the influences of different factors, such as types and heights of the structures, forces of earthquake and so on, but also keeps a simple and convenient static load type. We had also entrusted several colleges and research institutes with the dynamic analyses for the dam in addition. In these analyses and computations, the following assumptions had been adopted-

1. The concrete is a material with linear and isotropic elastic properties.
2. The foundation of dam is rigid.
3. The seismic wave is transmitted by the surface of foundation, where the earthquake acceleration of all kinds are alike everywhere.
4. The interaction problem between the structure and the fluid mass durning earthquakes, is consirdered starting from such assumption as the water in the reservoir is incompressible. The dynamic water pressure on the dam had been taken into account either as an APPARENT MASS of water or as a Simplified formula, developed by Westergard from a rigid dam with rectangular, vertical faces, retaining water. As for the earthquake-resistant analyses, two methods, called Vibration Modes Superposition and Response Spectra, had been used. Some seismograms, either recorded at home and abroad or artificial, had been used for seismograms input.

The following assumption had been used too-

1. As for the maximum acceleration of ground motions at the dam site, 0.2 g had been used on account of that the seismic intensity we used in the design is VIII, or about of magnitude 6 in Richter's Scales, and 0.4 g had been used to check safety of dam, by this time the earthquake intensity is IX.

2. As for the criteria of allowable stresses, during earthquake, we adopt a 30% increase in maximum allowable compressive stress and tensile stress for concrete over that for the usual loading combinations.

Please give comments on and suggestions for following problems:

1. Evalution of every results obtained from different analysis methods.

2. Evaluation of the dynamic parameters used in the computations and analyses, please give suggestions for improvement.

3. Please introduce the U.S. dynamic analysis methods and experiences. We would like to make a checking computation for this dam too, using the general dynamic analysis methods of the U.S., and to contrast the result with ours obtained from different methods.

4. How to deal with the influence of hydrodynamic pressure in dynamic analyses?

5. How to consider the flexibility of founctions of dam in dynamic analyses?

6. What are the criteria of allowable stresses for concrete dams during earthquake?

7. If an intensity of IX which is samilar with maximum credible earthquake (MCE), is used to make a checking computation, what are the maximum allowable stresses?

8. How to consider the dynamic stability of the rock masses at the dam abutments?

V. Main problems on construction planning

1. River diversion

(1). Recurrence period of designed flood.

(2). Practicability of the two alternate diversion schemes--one with overflow cofferdams and the other with non-overflow cofferdams, as presented in our work.

Suggestion for further improvement of diversion scheme.

(3). General layout and design of diversion works including tunnels and cofferdams. Important technical problems which deserve careful consideration in the course of designing these structures.

2. Construction technique

(1). Method of excavation along the high banks (> 300 m.) of both abutments and selection of main equipment. Due to the high pressure of in-situ stress as characterized by the frequent "cake-like" ruptures in the drill cores, effective measures is be taken during the excavation process to insure safety and the finished abutment being in satisfactory condition.

Please give some examples in U.S.A. and other countries.

(2). Stability of rock mass between under ground cavities (caverns) closely situated in area of high in situ rock stress. Safety considerations in the course of construction. Give opinions as for the construction procedure construction method and important technical measures which are necessary for speeding up construction.

(3). Method of dumping and placing concrete, selection of main equipment and its capacity and other effective measures that will help speed up dam construction.

(4). General arrangement of aggregate processing plant, (producing both cracked stone and crushed sand) batching and mixing plants of concrete, transportation of concrete etc.

VI. Summary of rock mechanic tests and main problems

A great deal of rock mechanic tests, in situ as well as laboratory, have been completed in the dam site area of Ertan Hydroelectric Power Station. Among them there are large scale, field direct shear tests 28 sets (surface stress 2 sets, three-dimension stress 13 sets, plan stress 6 sets, stress in deep drill holes in the river-bed 2 sets); cement and chemical grouting tests 2 sets, 22 bore holes, 16 sections; laboratory medium scale shear tests 82 sets; indoor elastic modulus tests 133 sets; seismic wave velocity measurements in adits 4896 m; acoustic wave measurements in adits 1598 m; microscope identification of polished rock sections 288 sets; chemical analysis of rock minerals and X-ray analysis of soft minerals 35 sets. Through these tests mentioned above and considering the geologic conditions of the dam site area, suggested values of rock mechanic parameter, needed in design have been determined. Further study for confirming is continuing. The mains of them are down hole deformation modulus measurements and creep tests of uralitized basalt, double shear tests of syenite and shear tests of some clay fillings, contraction measurements of the chambers in the underground powerplant locations, and grouting tests for strengthening the inferior rocks.

Please give comments on and suggestions for the following problems:

1. The methods of some main tests and evaluation of their results, the improvement for further tests.

2. Determination of the shearing strength of the rock in the foundations of main structures, including rocks in the dam abutments, country rock of the underground powerplant and the inferior rocks, such as the uralitized basalt in the right abutment.

We consider the determination of these parameters not only depends on the representativeness of the tested rock units but also the direction and magnitude of the force acting upon the rock in the foundation of the structure concerned and it also relates to the method of analysis as well as the safty factor used in the design. We listed the results of the tests. Please determine the correspondent design parameters according to the rationale and experience of geologic and rock mechanic investigations in the United States. For example if the rigid block method is exployed to study the stability of rock abutments, what is the safty factor? What rock mechanic parameters should be used, the arithmetic average, the low value or the average of the low values? How to combine the results of field tests with that of laboratory tests? Please determine the shearing strength of the rocks and its suggested values in accordance with our test results and the following conditions:

(1). When three-dimensional rigid block method is used to study the stability of rock abutments.

(2). In three-dimensional or two-dimensional finite element method, how to determine the shearing strength and the safty limit of iamage?

(3). In the stability analysis of the country rock of a chamber, how to determine the shearing strength of the rock, its allowable tensile stress and the safety factor used in the analysis?

3. How to determine the shearing strength of the rock with discontinuous joints?

4. How to determine the deformation moduli of the subregions in the dam abutments according to the geologic subdivision and the test data?

Please evaluate our analytical results and give suggestions for improvement.

5. Evaluation of the engineering mechanic properties of the uralitized basalt zones.

(1). Please evaluate the existing down hole test results (there are three methods) and give the suggested values of their deformation moduli.

(2). How to evaluate the shearing strength of these rock masses? What supplemental in situ shear tests should be done? How to determine the shearing strength used in analysis? Please give your suggestions on the test methods and techniques.

(3). Under confined and enclosed condition but with action of seepage flow, how will the engineering mechanic properties of these uralitized basalt zones change? What supplemental investigations can confirm these changes?

(4). What are the effective treatments for the uralitized basalt to improve its engineering mechanic properties?

6. In-situ stress measurements and application of their results

(1). How to measure in-situ stresses in the jointed rock mass?

(2). The methods of measurement, we used at present only adapt ot rather intact rock. Because of the high elastic modulus of the intact rock, the results of measurement are on the high side. They can not represent the deformation property of the jointed rock masses. How to modify the results?

(3). The difference between the in-situ stresses in the rock measured with the same method at different locations in a same region or that measured with different methods in the same place is rather great. How to determine the reasonable in situ stress parameters from these fifferent results?

(4). How to determine the three dimensional in situ stress field in the location of the structure base on a few measured data?

7. Questions of design and construction of the structures in high in situ stress location

(1). Is it necessary to consider the effects of the in situ stress in the stability analysis of rock abutments? How to consider it if necessary?

(2). When field tests are carried out in the location of high in situ stress, the natural rock stresses are often relieved and the rock mass is relazed during excavating the adit and preparing the test specimen. It certainly affects the measured mechanic parameters of the rock in some degree. How to improve the test method and technique in order to find out the effect of in situ stresses?

(3). "Poker chip" disking in many intervals of rock core, particularly the syenite, is found. Usually it occures in the place of higher in situ stresses. Is it necessary to take some protection measures during excavation of the dam foundation? What are the effective protection measures if necessary? In the past during adit driving, rock fall due to relief of in situ stresses took place in some locations. How to assess the risk of rock brust and its intensity in the future? What monitoring system should be adopted in the construction of underground structures? Can rock bolt and shotcrete or other measures prevent rock burst?

8. Design and consturction of large size chambers

(1). In the location of the underground powerplant in the left abutment, the rock is rather intact with sparse joints which are closed. Can such rock masses be regarded as linear elastic body in the finite element method analysis of country rock of the powerplant?

(2). In the analysis of stability of country rock, we usually use finite element method to determine the scope of possible damage and unbalance of the rock through study of the stresses and deformation in it, and to find out the stresses and deformation of the rock as a result of stress relief during excavation. By these we estimate the condition of the country rock and degree of its unbalance. Are there any experimental criteria to assess the condition of the rock in excavation of large size chambers?

The results of shear tests on rock and the suggested shearing strength

Shear types	Rocks	Typical type		No. of test point	Shear off lgα	Shear off b	Shear off points/sets	Shearing strength b	Shearing strength points/sets	Arithmetic average	Low value average	number of joints	Suggested values lgα	Suggested values b
Rock to rock shear off	Syenite	Weakly to slightly weathered, blind joints are rather developed		τ5-4	2.1	25.0	6/1	35.5	1					
		slightly weathered, blind joints are not very developed		τ5-6	2.4	37.0	4/1	32.03	1					
		Rock is fresh with good integrity, most of the joints are closed without fillings. During tests intact rock was sheared off.		τε 4-2-7				25.7						
				τε 4-2-8				35.4						
				τε 4-2-9				40.3						
				τε 4-2-10				59.7	8/2	43.94	34.02	11	1.2	30.0~34.0
				τε 4-3-1				26.95						
				τε 4-3-2				63.10						
				τε 4-3-3				42.29						
				τ6 4-3-4				59.78						
				τε 4-3-5				62.62						
		yield limit		τε5-6	1.2	30.0	5/1							
	Altered basalt	Porphyritic altered basalt, uneven in lithofacies, weakly to slightly weathered with bad integrity. Blind joints are rather developed. During tests rock was sheared off along preexisting joints in various degree	Joints are developed, rust stain widespread, some joints are filled with mud film or clay.	τε 1-1	1.55	3.0	6/1	2.19	1			1	0.8	10.0
			Joints are developed, their surfaces are rather fresh, part of them are filled with chlorite film.	τε 1-4	1.44	23.0	5/1							
			Rock is rather intact. Its surface is fresh only very few joints with rust stain. calcium film or chlorite film	τε 1-3				[redacted] 3.0 30.82	3/1	20.0	3.0	3		
		Rock is fresh with good integrity even in lithofacies											1.0	20.0
	Second stratum basalt	Slightly weathered—fresh with bad integrity. most joints are of medium to high angle and are filled with chlorite or calcium film, during tests rock was sheared off mainly along blind—micro joint, most specimens were crushed.		τε 30-1	1.9	21.0	7/1	11.30	1			1	0.8	8.0
	Unutilised basalt	Fresh with bad integrity. There are many blind—micro joints, short and irregular, commonly filled with talc, chlorite and soapstone film. During tests rock was sheared off basically along blind joints. The portion of intact rock sheared off made up 5-10%		τε 30-3	1.52	12.5	7/1	6.07	1			1	0.05~0.50	0
	Third stratum basalt	Weakly to slightly weathered. Joints are developed. The joints are commonly filled with calcite. There are rust stain of various degree on the joint surfaces. During tests rock was sheared off along the joints of medium—high angle. The sheared surfaces were irregular in form		τ13-1	1.82	30.0	6/1			40.8 (high value)	20 (low value)	2	1.0	25.0

(3). In the finite element method analysis of underground openings, how to estimate the effects of rock bolt and shotcrete? In engineering practice rock bolt and shotcrete are very effective. How to analyse their effects quantitively?

(4). Please pass on experience of design and construction of rock bolt and shotcrete with some practical projects.

(5). Please recommend the monitering systems and monitering survey in construction of chambers with some practical projects.

9. Others

(1). In rock foundation the seepage water flows mainly through the joints, so it is heterogenous. Darcy's Law for seepage flow in homogeneous medium V=KJ (where k is the coefficient of percolation can hardly be used in such a case. How do you determine the field of flow net under this condition?

(2). When anchor lines are used to strengthen the rock abutments of an arch dam, how to determine the force in the anchorage cables? Which method of analysis is better to find out the force, the rigid block method or the finite element method?

VII. Turbine-generator sets, gates and hoists

1. Turbine-generator sets

You are requested to describe the following topics:

(1). The operating reliability of large hydrogenerating sets (including the Grand Coulee large generating units from 1980 to 1983) each with a capacity of more than 400 MW and the causes for serious accidents.

(2). Your suggestion for the optimum turbine parameters (Q_t', N_t', Ns, σ , η) suitable to the Ertan Project.

(3). The method of calculating the turbine regulation warranty and its relevant rule, specifications and instructions.

(4). The technology of design, calculation and construction methods for combined strength of spiral case with reinforced concrete assistance.

(5). The means to prevent the Ertan turbine from cavitation.

The anti-cavitation materials used such as for Ertan turbine. The experiences of aeration into the location of cavitation and erecting fins in draft tube.

(6). The design and operation of a long (more than 10 times of turbine diameter) and narrow (1.5-2 times of turbine diameter) draft tube.

(7). The design of the large size spiral case near for the Ertan turbine and the result of its hydraulic test (the hydraulic loss, the flow distribution, etc.).

(8). For a power plant with a 150-200 m. head, what would be the most economical and effective measure for water supply to unit cooling, station service and fire protection systems?

(i). If a jet pump is used, please introduce its operating condition (noise, cavitation, efficiency and discharge).

(ii). If directly the low pressure water coming through the labyrinth ring is used, how the operational condition will be? Will it influence the operating stability of runner (increase vibration or swing)?

2. Gates and hoists

(1). The recent design development in gates with high head and large sizes and in hoists with large capacity.

The code, specification and instruction adopted by Corps of Engineers, U.S. Army and Bureau of Reclamation in the selection of types of gate, and in the structural and hydraulic computation of the gates.

(2). The working gate for the middle level outlet of Ertan Project has a size of 5m x 6m (w x h), its design head is about 100m. In order to seal water tightly, radial gate with eccentrical shaft trunnion is suggested. Please introduce an example of a project, the design considerations, result of test and operation experiences of the eccentrical trunnion radial gate with a head of 100 m or above.

The emergency gate is suggested to use slide gate of 5.0m x 7.5m (w x h). What are the low friction and high strength bearing materials?

If caterpillar gate to be adopted, please tell us its design and operation experiences.

(3). The working gate of the spillway tunnel has an opening of 11m x 10m (w x h), its design head is 75 m. The total water pressure is 9000 tons. The type of gate is radial. In order to reduce the thrust carried by each trunnion, it is suggested to arrange 4 trunnions. If you ever met the similar, tell us its design and operational condition.

(4). The velocity in the spillway tunnel and in the middle level outlet is nearly 35 M/Sec. Suitable forms of gate slot are determined by model tests. But what are the design criteria to decide the liner, the selection of liner materials and the measure to eliminate cavitation?

(5). Illustrate us the prestressed anchorage of a large taintor gate (16x17.5m or 16x11.5m) with an engineering example. The results in lab test and of prototype test.

(6). The type of hoist commonly used for large capacity (say 2x120 T. and above) and for large sizes (say 16m x 16m and above) radial gate.

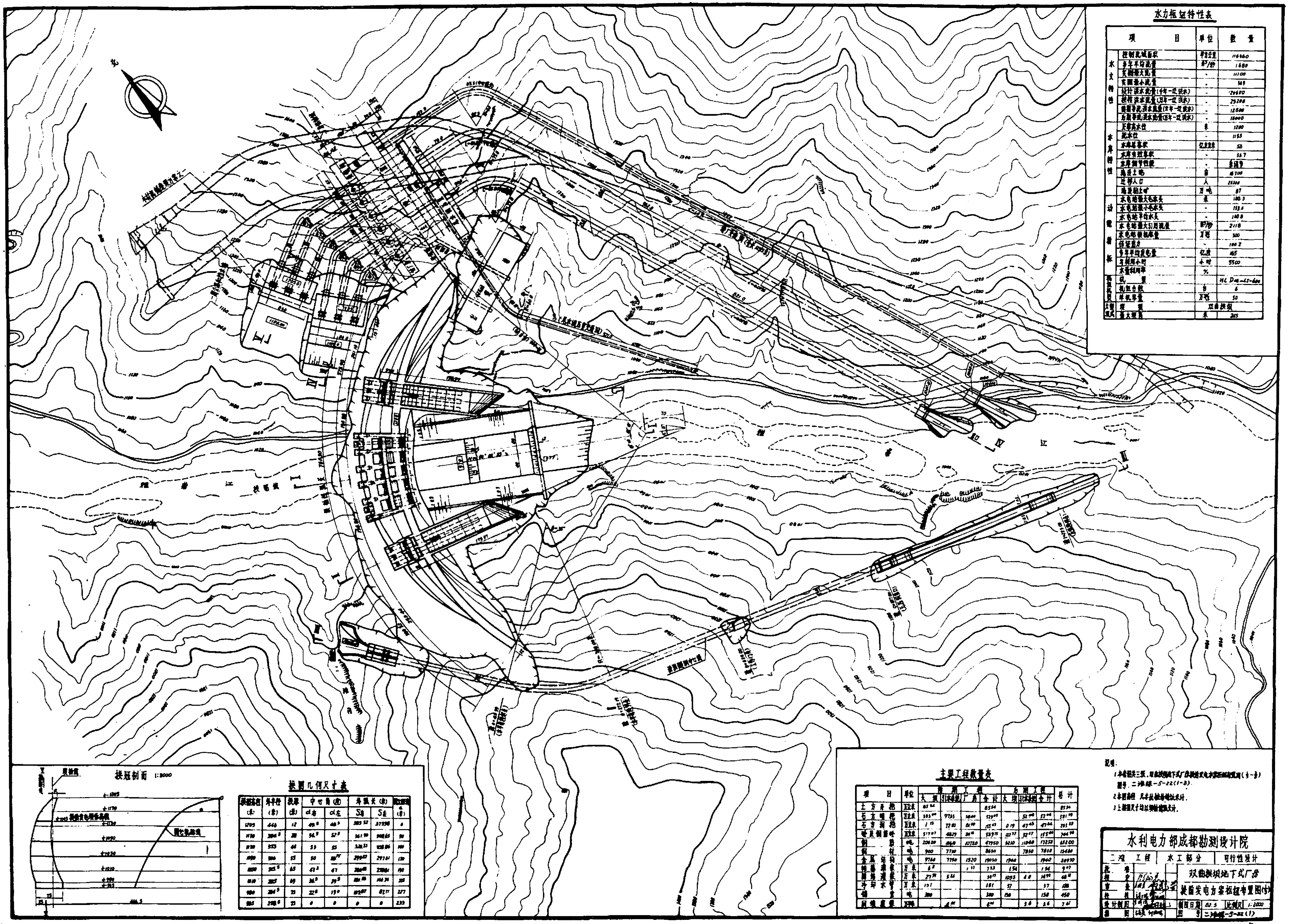
(7). In Ertan Project, there are 800-900 thousands M^3 of logs needed to pass the dam in July-September every year. During this time, the reservoir level fluctuation is in a range of 25m. Mean diameter of the log is 0.3m and its mean length is 4.5m. It is suggested to pass logs mechanically by longitudinal chain conveyer with a capacity of 4000 logs per hour.

Please introduce the general layout of the chain conveyer, the feeding equipment and the type of hydraulic accelerator, and the means to harmonize the inlet mouth with the reservoir level fluctuation.

APPENDIX F

DRAWINGS*

* Reproduced by photocopy from a report entitled, "BRIEF DESCRIPTION OF ERTAN HYDROELECTRIC PROJECT DESIGN" by Chengdu Hydroelectric Power Survey and Design Institute, Chengdu, China, November 1982.



水力枢纽特性表

项目		单位	数量
水文特性	控制流域面积	平方公里	116360
	多年平均流量	米³/秒	1680
	实测最大流量	-	11100
	实测最小流量	-	349
	设计洪水流量(千年一遇洪水)	-	20600
	校核洪水流量(万年一遇洪水)	-	25200
	施工导流洪水流量([illegible]年一遇洪水)	-	12500
	后期导流洪水流量(百年一遇洪水)	-	16000
水库特性	正常高水位	米	1200
	死水位	-	1155
	水库总库容	亿立方米	58
	水库有效库容	-	33.7
	水库调节性能	-	季调节
	淹没土地	亩	18700
	迁移人口	人	21300
	[illegible]	万吨	87
动能指标	水电站最大毛水头	米	180.3
	水电站最小毛水头	-	133.[illegible]
	水电站平均水头	-	140.8
	水电站最大引用流量	米³/秒	2118
	水电站装机容量	万瓩	300
	保证出力	-	100.2
	多年平均发电量	亿度	165
	年利用小时	小时	5500
	水量利用率	%	
机组规范	机型		HL D[illegible]-LJ-600
	机组台数	台	6
	单机容量	万瓩	50
大坝规范	坝型		双曲拱坝
	最大坝高	米	245

拱冠剖面 1:3000

拱圈几何尺寸表

拱圈高程 (米)	外半径 (米)	拱厚 (米)	中心角(度) α右	α左	弧长(米) S右	S左	[illegible] (米)
1205	446	12	49.[illegible]	46.[illegible]	383.52	375.96	0
1170	384.[illegible]	28	56.[illegible]	52.[illegible]	361.90	348.65	90
1130	353	44	59	55	326.33	[illegible]	110
1090	[illegible]	53	58	[illegible]	299.[illegible]	297.[illegible]	130
1050	[illegible]	65	47.3	47	[illegible]	[illegible]	[illegible]
1010	285	69	36.[illegible]	29.5	[illegible]	146.34	[illegible]
980	284.5	73	22.6	19.0	[illegible]	82.17	227
965	[illegible]	73	0	0	0	0	233

主要工程数量表

项目	单位	前期工程 大坝	引水系统	厂房	合计	后期工程 大坝	引水系统	合计	总计
土方开挖	万立米	85.94			85.94				85.94
石方明挖	万立米	393.00	97.65	38.40	529.05		52.06	52.06	581.11
石方洞挖	万立米	1.15	72.80	81.49	155.43	0.19	47.45	47.64	203.07
砼及钢筋砼	万立米	517.07	48.29	31.[illegible]	597.[illegible]	122.77	32.47	155.[illegible]	744.90
钢筋	吨	20620	[illegible]	10720	47950	6210	11040	17250	65200
钢材	吨	900	7730		8630		7810	7810	16440
金属结构	吨	9760	7750	1520	19030	1940		1940	20970
帷幕灌浆	万米	6.2		1.11	7.33	1.54		1.54	8.87
固结灌浆	万米	27.[illegible]	5.[illegible]		34.17	10.55	4.4	14.95	48.[illegible]
冷却水管	万米	151			181	37		37	[illegible]
[illegible]	吨	300			300	150		150	450
回填灌浆	[illegible]		4.[illegible]		4.[illegible]		3.6	3.6	7.61

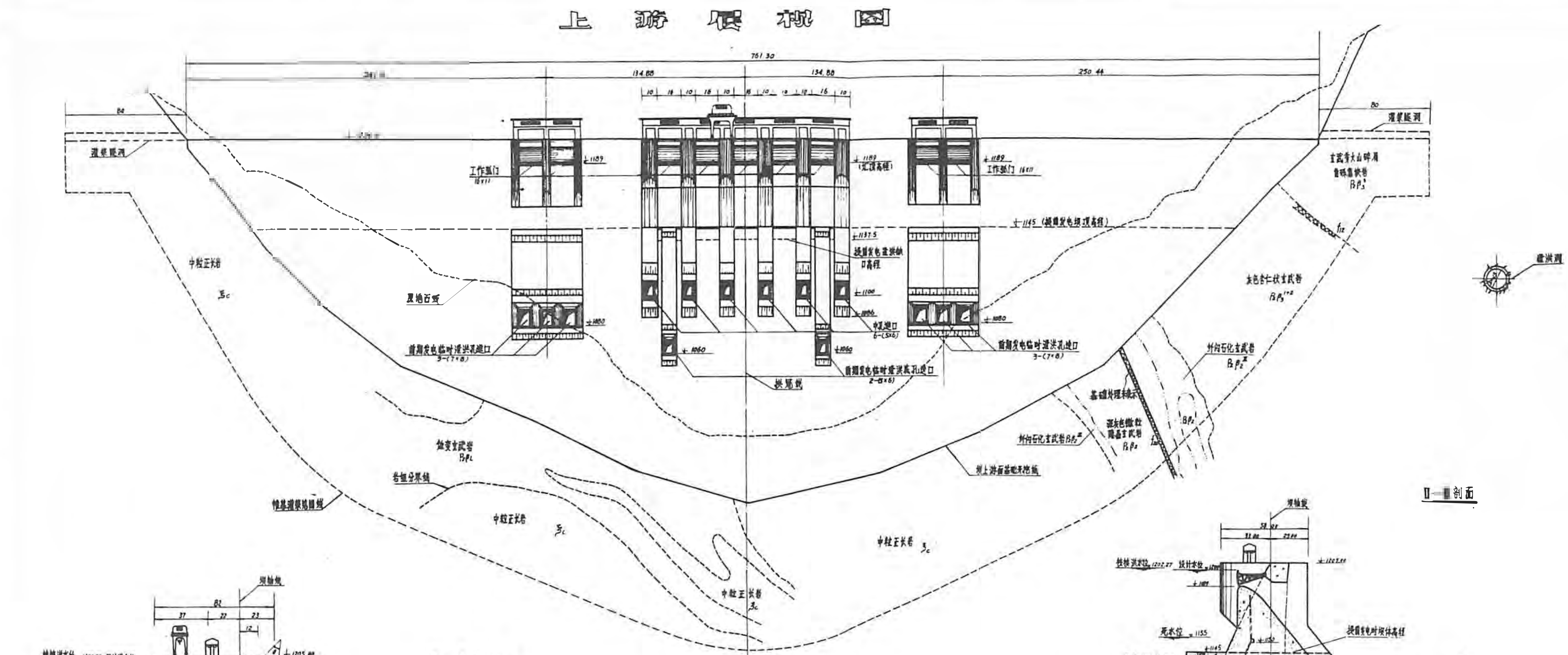
说明:

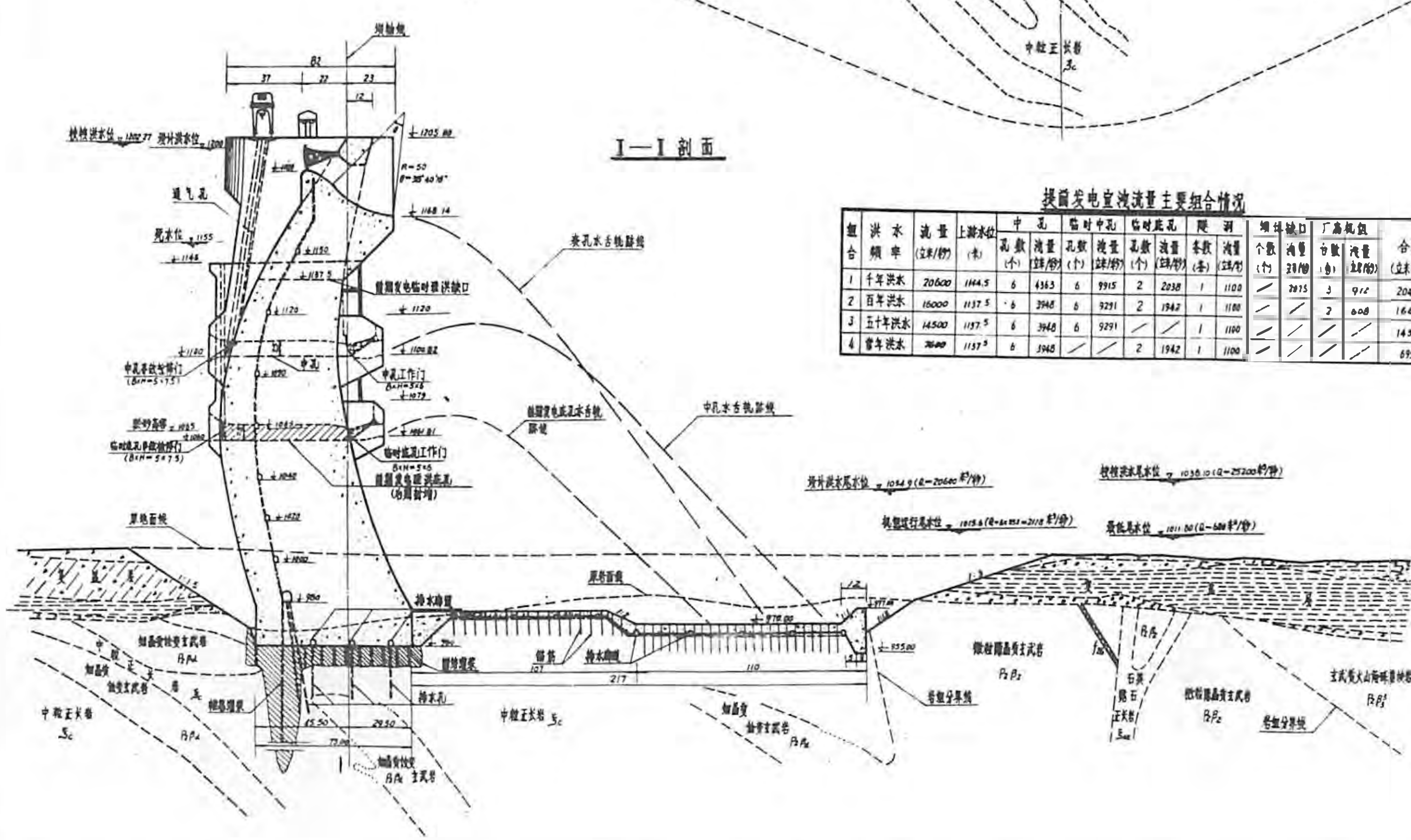
1 本套图共三张，双曲拱坝地下式厂房[illegible]方案枢纽布置图(3-3) 图号 二滩水綫-5-22(1-3).

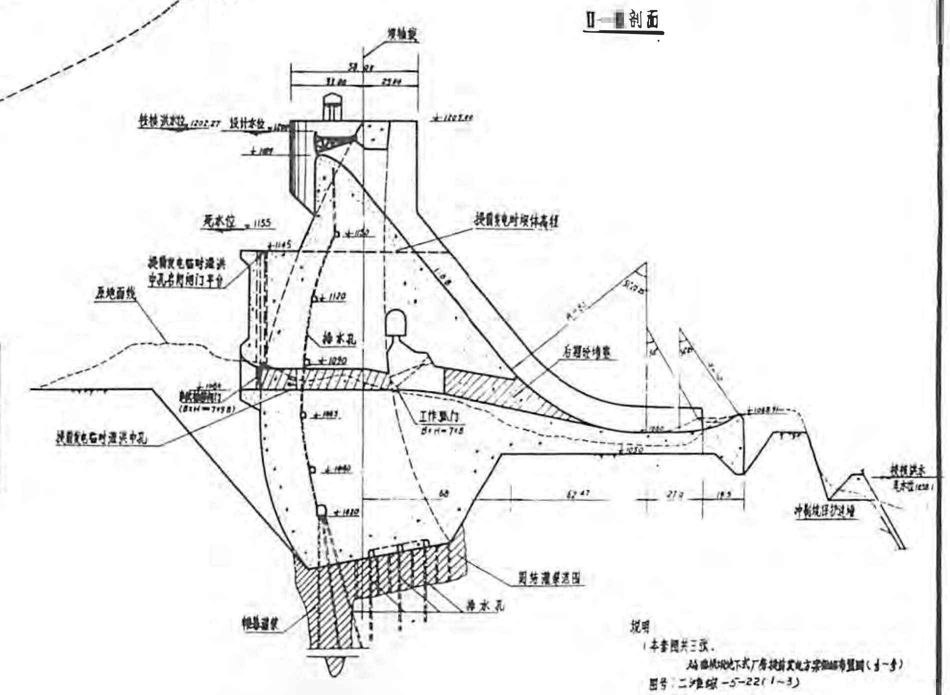
2 本图高程 [illegible].

3 [illegible].

水利电力部成都勘测设计院		
二滩 工程	水工部分	可行性设计
批准 / 核定 / 审查 / 校核		双曲拱坝地下式厂房 左岸发电方案枢纽布置图(1)
设计制图		制图日期 82.5 比例尺 1:2000
描图		图号 二滩水綫-5-22(1)







提前发电宣泄流量主要组合情况

组合	洪水频率	流量（立米/秒）	上游水位（米）	中孔 孔数（个）	中孔 流量（立米/秒）	临时中孔 孔数（个）	临时中孔 流量（立米/秒）	临时底孔 孔数（个）	临时底孔 流量（立米/秒）	隧洞 条数（条）	隧洞 流量（立米/秒）	坝体缺口 个数（个）	坝体缺口 流量（立米/秒）	厂房机组 台数（台）	厂房机组 流量（立米/秒）	合计（立米/秒）
1	千年洪水	20600	1144.5	6	4363	6	9915	2	2038	1	1100	/	2015	3	912	20401
2	百年洪水	16000	1137.5	6	3948	6	9291	2	1942	1	1100	/	/	2	608	16889
3	五十年洪水	14500	1137.5	6	3948	6	9291	/	/	1	1100	/	/	/	/	14339
4	常年洪水	7640	1137.5	6	3948	/	/	2	1942	1	1100	/	/	/	/	6990

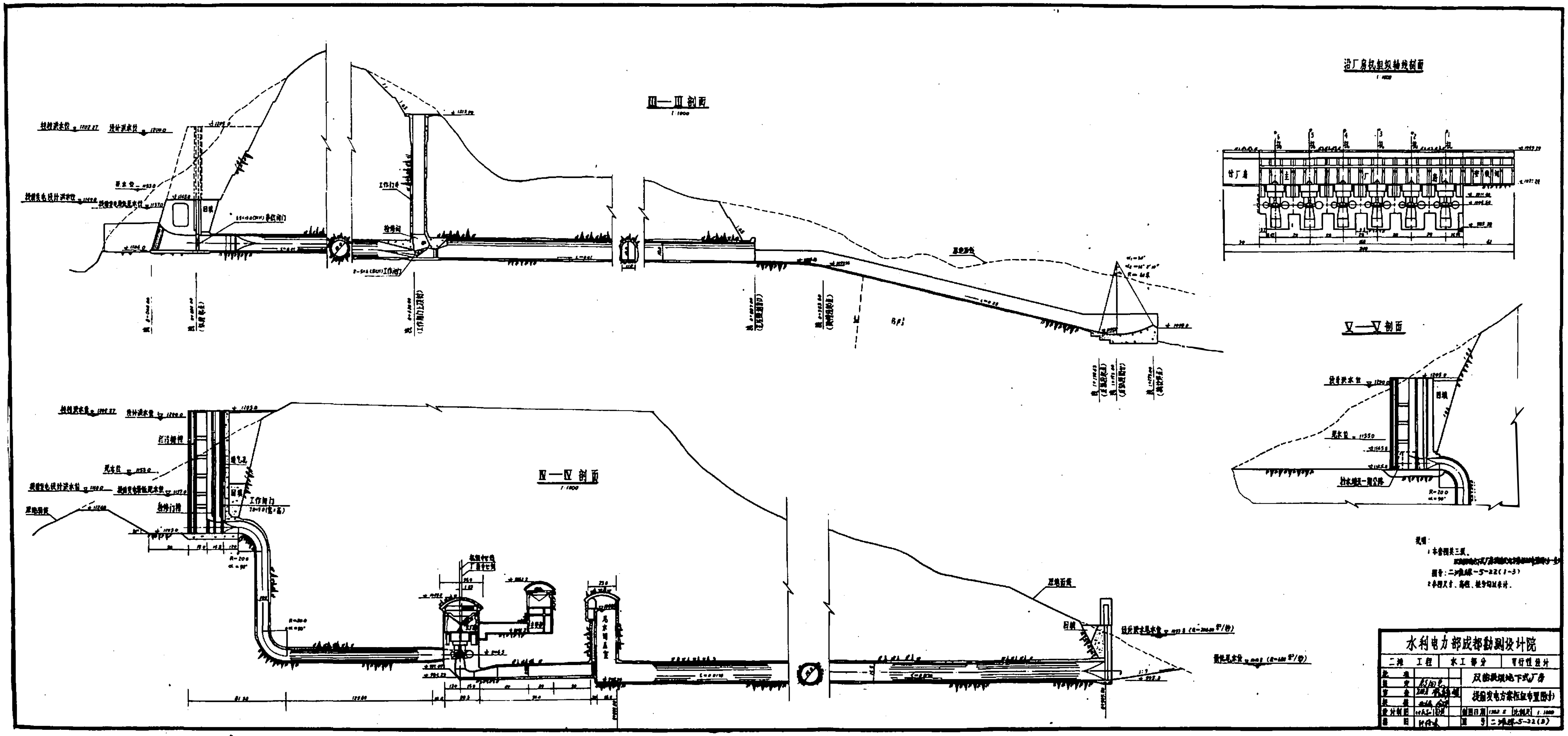
枢纽宣泄流量主要组合情况

组合	洪水频率	流量（立米/秒）	上游水位（米）	[illegible]底孔 孔数（个）	[illegible]底孔 流量（立米/秒）	开敞[illegible]式 孔数（个）	开敞[illegible]式 流量（立米/秒）	中孔 孔数（个）	中孔 流量（立米/秒）	隧洞 条数（条）	隧洞 流量（立米/秒）	厂房机组 台数（台）	厂房机组 流量（立米/秒）	合计（立米/秒）
1	万年洪水	29200	1221.17	5	8310	4	6600	6	8372	1	2000	6	2118	29200
2	千年洪水	20500	1201.[illegible]	5	6072	4	4858	6	8372	1	2000	4	1412	20714
3	百年洪水	16000	1200.00	5	6072	/	/	6	6072	1	2000	4	1412	15856
4	常年洪水	7600	1200.00	5	6072	/	/	/	/	/	/	4	1412	7484

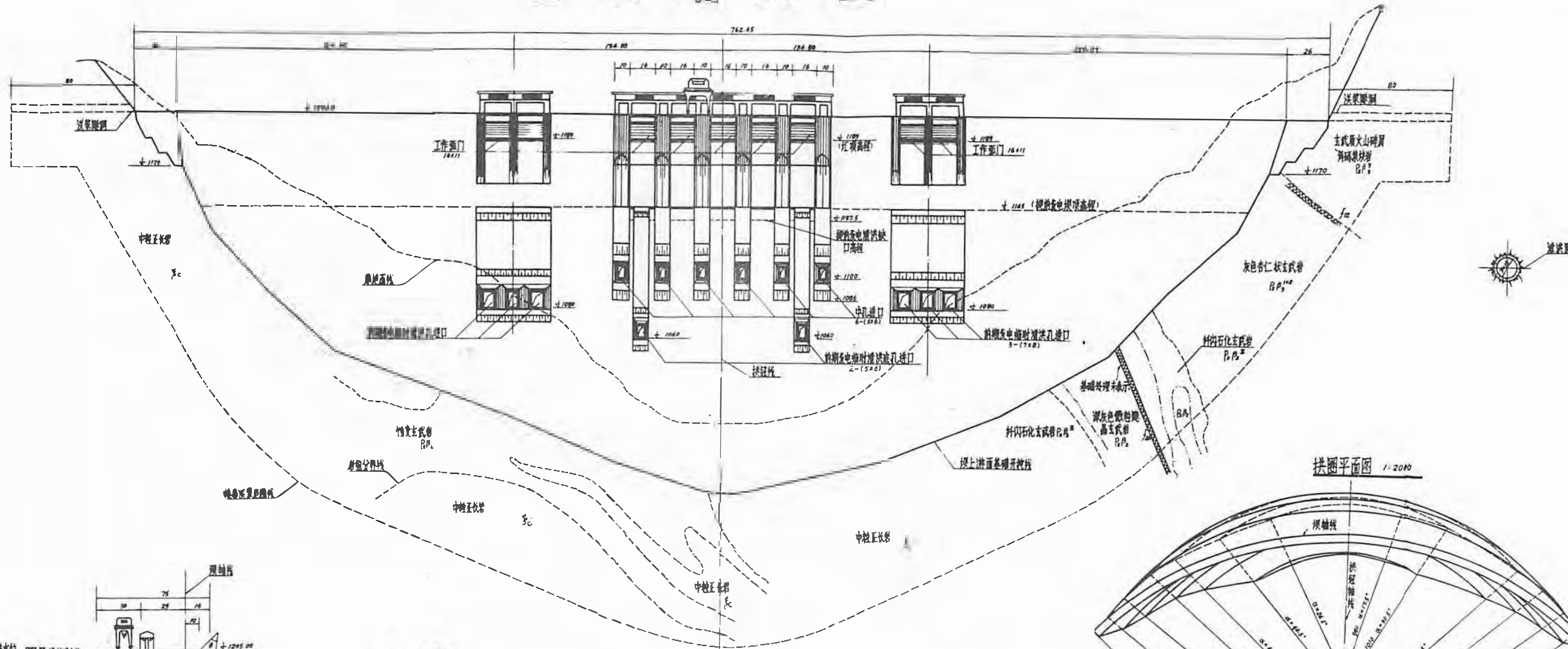
说明

1. 本套图共三张。
   双曲拱坝地下式厂房提前发电方案枢纽布置图（1/3—3/3）
   图号：二滩水坝—5—22（1—3）
2. 本图尺寸、高程均以米计。
3. 坝顶所注尺寸除溢流段为闸门控制处的尺寸，其余均为坝轴线的弧长。

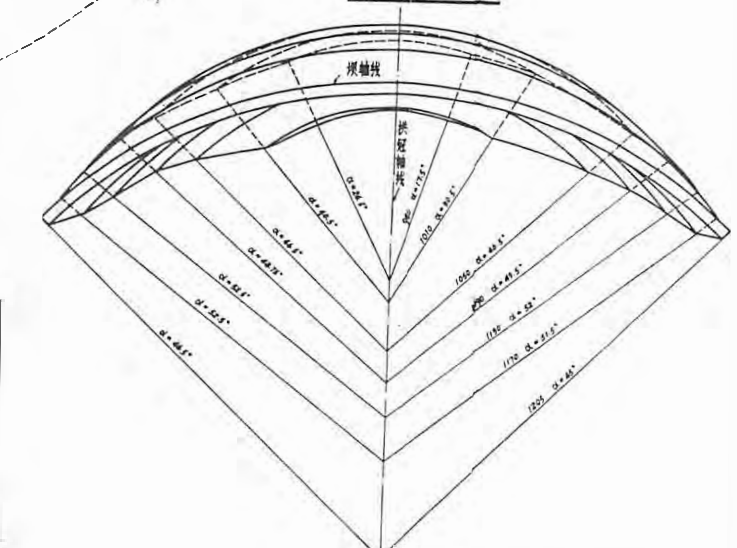
水利电力部成都勘测设计院		
二滩工程	水工部分	可行性设计
批准		双曲拱坝地下式厂房
核定		提前发电方案枢纽布置图（2/3）
审查		
校核		制图日期 82.5 比例尺 1:1000
设计制图		
描图		图号 二滩水坝—5—22（2）


水利电力部成都勘测设计院
二滩 工程
水工 部分
可行性设计
地下式厂房
二滩EK-5-22(1-3)

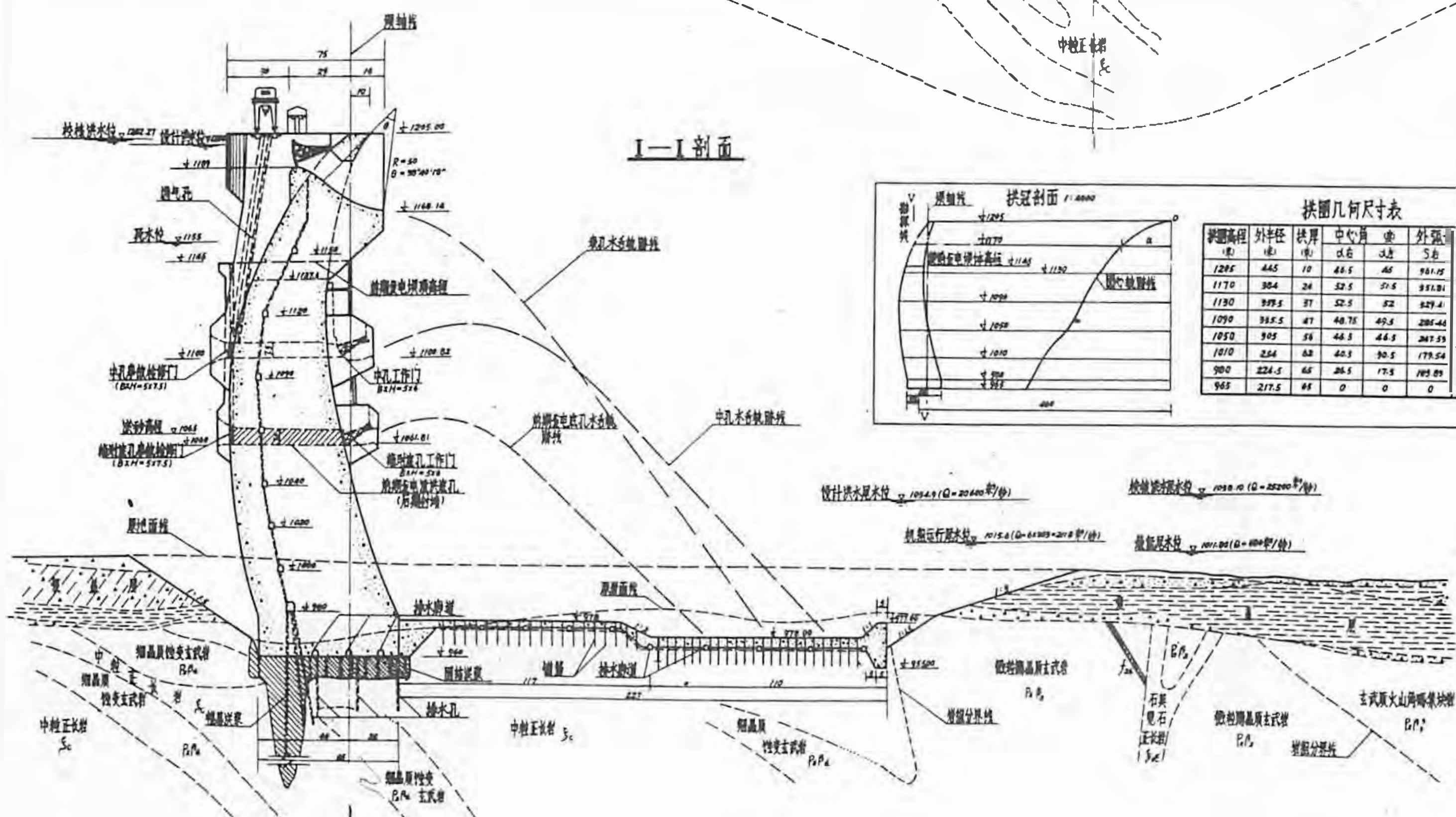
上游立视图



拱圈平面图 1:2000



I—I剖面



拱圈几何尺寸表

拱圈高程 (m)	外半径 (m)	拱厚 (m)	中心角(度) 左	中心角(度) 右	外弧长(m) 左	外弧长(m) 右	距
1205	445	10	46.5	46	361.15	361.50	0
1170	384	24	52.5	51.5	351.81	341.16	40.0
1130	357.5	37	52.5	52	329.4[illegible]	326.57	138.8
1090	335.5	47	48.75	49.5	285.46	289.86	164.8
1050	305	56	46.5	46.5	247.59	241.60	194.0
1010	254	62	40.5	40.5	179.54	190.21	240.0
980	224.5	65	26.5	17.5	103.83	68.57	234.5
965	217.5	65	0	0	0	0	247.5

主要工程量表

项目	单位	一期工程 大坝	一期工程 引水系统	一期工程 厂房	一期工程 合计	二期工程 大坝	二期工程 引水系统	二期工程 合计	总计
覆盖层开挖	万立米	85.54			85.54				85.54
石方明挖	万立米	350	[illegible]	[illegible]	496.00		52.04	52.[illegible]	548.09
石方洞挖	万立米	[illegible]	72.8[illegible]	[illegible]	155.83	0.19	87.46	47.64	203.39
砼及钢筋砼	万立米	470.34	60.29	20.16	560.78	111.77	32.67	144.44	690.32
钢筋	吨	180.90	14410	10780	44100	5760	11060	16800	61900
钢材	吨	900	7730		8600		7800	7800	[illegible]
金属结构	吨	5760	7740	1080	17000	1996		1998	20000
帷幕灌浆	万米	6.3		[illegible]	7.80	1.54		1.54	9.[illegible]
固结灌浆	万米	27.9	5.2[illegible]		39.17	10.95	4.0	14.95	44.48
冷却水管	万米	151			151	37		37	18[illegible]
钢管	吨	300			300	150		150	450
回填灌浆	万平米		4.41		4.[illegible]		5.6	5.6	7.[illegible]

说明：

1. 本方案枢纽平面布置及其它剖面参见双曲拱坝地下式厂房提前发电方案枢纽布置图（5—9）图号：二滩可—5—22（1—3）
2. 本图尺寸高程均以米计。
3. 除图中注明比例外，各图比例均为1:1000

水利电力部成都勘测设计院			
二滩工程	工 部分	可行性设计	
批准		双曲拱坝地下式厂房提前发电改进方案剖面图	
核定	[illegible]		
审查	[illegible]		
校核	[illegible]		
设计制图	[illegible]	制图日期	1982.10 比例尺
描图	[illegible]	图号	二滩可—5—22（11）