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SUSITNA HYDROELECTRIC PROJECT

FERC LICENSE APPLICATION

EXHIBIT B

FIRST DRAFT

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ALASKA POWER AUTHORITY

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EXHIBIT B - PROJECT OPERATION AND RESOURCE UTILIZATION

1 - DAMSITE SELECTION

This section summarizes the previous site selection studies and the studies done during the Alaska Power Authority Susitna Hydroelectric Project Feasibility Study. Additional detail on this topic can be found in the Development Selection Report, Reference 1.

1.1 - Previous Studies

Prior to the undertaking of the Susitna Hydroelectric Project Feasibility Study by the applicant, the hydroelectric development potential of the Alaskan Railbelt had been studied by several entities.

(a) Early Studies of Hydroelectric Potential

Shortly after World War II ended, the United States Bureau of Reclamation (USBR) conducted an initial investigation of hydroelectric potential in Alaska and issued a report of the results in 1948. Responding to a recommendation made in 1949 by the nineteenth Alaska territorial legislature that Alaska be included in the Bureau of Reclamation program, the Secretary of Interior provided funds to update the 1948 work. The resulting report, issued in 1952, recognized the vast hydroelectric potential within the territory and placed particular emphasis on the strategic location of the Susitna River between Anchorage and Fairbanks as well as its proximity to the connecting Railbelt (see Figure B.1).

A series of studies was commissioned over the years to identify damsites and conduct geotechnical investigations. By 1961, the Department of the Interior proposed authorization of a two-dam power system on the Susitna River involving the Devil Canyon and the Denali sites (Figure B.2). The definitive 1961 report was subsequently updated by the Alaska Power Administration (an agency of the USBR) in 1974, at which time the desirability of proceeding with hydroelectric development was reaffirmed.

The Corps of Engineers (COE) was also active in hydropower investigations in Alaska during the 1950s and 1960s, but focused its attention on a more ambitious development at Rampart on the Yukon River. This project was capable of generating five times as much annual electric energy as the prior Susitna proposal. The sheer size and the technological challenges associated with Rampart captured the imagination of supporters and effectively diverted attention from the Susitna Basin for more than a decade. The Rampart report was finally shelved in the early 1970s because of strong environmental concerns and the uncertainty of marketing prospects for so much energy, particularly in light of abundant natural gas which had been discovered and developed in Cook Inlet.

The energy crisis precipitated by the OPEC oil boycott in 1973 provided some further impetus for seeking development of renewable resources. Federal funding was made available both to complete the Alaska Power Administration's update report on Susitna in 1974 and to launch a prefeasibility investigation by the COE. The State of Alaska itself commissioned a reassessment of the Susitna Project by the Henry J. Kaiser Company in 1974.

Although the gestation period for a possible Susitna Project has been lengthy, federal, state, and private organizations have been virtually unanimous over the years in recommending that the project proceed.

Salient features of the various reports to date are outlined in the following sections.

(b) U.S. Bureau of Reclamation - 1953 Study

The USBR 1952 report to the Congress on Alaska's overall hydroelectric potential was followed shortly by the first major study of the Susitna Basin in 1953. Ten damsites were identified above the railroad crossing at Gold Creek. These sites are identified on Figure B.2.

- Gold Creek;
- Olson;
- Devil Canyon;
- Devil Creek;
- Watana;
- Vee;
- Maclaren;
- Denali;
- Butte Creek; and
- Tyone (on the Tyone River).

Fifteen more sites were considered below Gold Creek. However, more attention has been focused over the years on the Upp-- Susitna Basin where the topography is better suited to dam construction and where less impact on anadromous fisheries is expected. Field reconnaissance eliminated half the original Upper Basin list, and further USBR consideration centered on Olson, Devil Canyon, Watana, Vee, and Denali. All of the USBR studies since 1953 have regarded these sites as the most appropriate for further investigation.

(c) U.S. Bureau of Reclamation - 1961 Study

In 1961 a more detailed feasibility study resulted in a recommended five-stage development plan to match the load growth curve as it was then projected. Devil Canyon was to be the first development--a 635- foot-high arch dam with an installed capacity of about 220 MW. The reservoir formed by the Devil Canyon dam alone would not store enough water to permit higher capacities to be economically installed, since long periods of relatively low flow occur in the winter months. The second stage would have increased storage capacity by adding an earthfill dam at Denali in the upper reaches of the basin. Subsequent stages involved adding generating capacity to the Devil Canyon dam. Geotechnical investigations at Devil Canyon were more thorough than at Denali. At Denali, test pits were dug, but no drilling occurred.

(d) Alaska Power Administration - 1974

Little change from the basic USBR-1961, five-stage concept appeared in the 1974 report by the Alaska Power Administration. This later effort offered a more sophisticated design, provided new cost and schedule estimates, and addressed marketing, economics, and environmental considerations.

(e) Kaiser Proposal for Development

The Kaiser study, commissioned by the Office of the Governor in 1974, proposed that the initial Susitna development consist of a single dam known as High Devil Canyon located on Figure B.2. No field investigations were made to confirm the technical feasibility of the High Devil Canyon location because the funding level was insufficient for such efforts. Visual observations suggested the site was probably favorable. The USBR had always been uneasy about foundation conditions at Denali, but had to rely upon the Denali reservoir to provide storage during long periods of low flow. Kaiser chose to avoid the perceived uncertainty at Denali by proposing to build a rockfill dam at High Devil Canyon which, at a height of 810 feet, would create a large enough reservoir to overcome the storage problem. Although the selected sites were different, the COE reached a similar conclusion when it later chose the high dam at Watana as the first to be constructed.

Subsequent developments suggested by Kaiser included a downstream dam at the Olson site and an upstream dam at a site known as Susitna III (see Figure B.2). The information developed for these additional dams was confined to estimating energy potential. As in the COE study, future development of Denali remained a possibility if foundation conditions were found to be adequate and if the value of additional firm energy provided economic justification at some later date.

(f) U.S. Army Corps of Engineers - 1975 and 1979 Studies

The most comprehensive study of the Upper Susitna Basin prior to the current study was completed in 1975 by the COE. A total of 23 alternative developments were analyzed, including those proposed by the USBR, as well as consideration of coal as the primary energy source for Railbelt electrical needs. The COE agreed that an arch dam at Devil Canyon was appropriate, but found that a high dam at the Watana site would form a large enough reservoir for seasonal storage and would permit continued generation during low flow periods.

The COE recommended an earthfill dam at Watana with a height of 810 feet. In the longer term, development of the Denali site remained a possibility which, if constructed, would increase the amount of firm energy available in dry years.

An ad hoc task force was created by Governor Jay Hammond upon completion of the 1975 COE Study. This task force recommended endorsement of the COE request for Congressional authorization, but pointed out that extensive further studies, particularly those dealing with environmental and socioeconomic questions, were necessary before any construction decision could be made.

At the federal level, concern was expressed at the Office of Management and Budget regarding the adequacy of geotechnical data at the Watana site as well as the validity of the economics. The apparent ambitiousness of the schedule and the feasibility of a thin arch dam at Devil Canyon were also questioned. Further investigations were funded and the COE produced an updated report in 1979. Devil Canyon and Watana were reaffirmed as appropriate sites, but alternative dam types were investigated. A concrete gravity dam was analyzed as an alternative for the thin arch dam at Devil Canyon and the Watana dam was changed from earthfill to rockfill. Subsequent cost and schedule estimates still indicated economic justification for the project.

1.2 - Plan Formulation and Selection Methodology

The proposed plan which is the subject of this license application was selected after a review and reassessment of all previously considered sites. Additional detail in support of the findings in this Exhibit is found in Reference 5.

This section of the report outlines the engineering and planning studies carried out as a basis for formulation of Susitna Basin development plans and selection of the preferred plan.

In the description of the planning process, certain plan components and processes are frequently discussed. It is appropriate that three particular terms be clearly defined:

<u>Damsite</u> - An individual potential damsite in the Susitna Basin, referred to in the generic process as "candidate."

<u>Basin Development Plan</u> - A plan for developing energy within the Upper Susitna Basin involving one or more dams, each of specified height, and corresponding power plants of specified capacity. Each plan is identified by a plan number and subnumber indicating the staging sequence to be followed in developing the full potential of the plan over a period of time.

<u>Generation Scenario</u> - A specified sequence of implementation of power generation sources capable of providing sufficient power and energy to satisfy an electric load growth forecast for the 1980-2010 period in the Railbelt area. This sequence may include different types of generation sources such as hydroelectric and coal, gas or oil- fired thermal. These generation scenarios were developed for the comparative evaluations of Susitna Basin generation versus alternative methods of generation.

In applying the generic plan formulation and selection methodology, five basic steps are required; defining the objectives, selecting candidates, screening, formulation of development plans, and, finally,, a detailed evaluation of the plans (see Figure B.3). The objective is to determine the optimum Susitna Basin development plan. The various steps required are outlined in subsections of this section.

Throughout the planning process, engineering layout studies were made to refine the cost estimates for power generation facilities or water storage development at several damsites within the basin. These data were fed into the screening and plan formulation and evaluation studies.

The second objective, the detailed evaluation of the various plans, is satisfied by comparing generation scenarios that include the selected Susitna Basin development plan with alternative generation scenarios, including all-thermal, and a mix of thermal plus alternative hydropower developments.

1.3 - Damsite Selection

In previous Susitna Basin studies, twelve damsites were identified in the upper portion of the basin, i.e., upstream from Gold Creek. These sites are listed in Table B.1 with relevant data concerning facilities, cost capacity, and energy.

The longitudinal profile of the Susitna River and typical reservoir levels associated with these sites are shown in Figure B.4. Figure B.5 illustrates which sites are mutually exclusive, i.e., those which cannot be developed jointly, since the downstream site would inundate the upstream site.

All relevant data concerning dam type, capital cost, power, and energy output were assembled and are summarized in Table B.1. For the Devil Canyon, High Devil Canyon, Watana, Susitna III, Vee, Maclaren, and Denali sites, conceptual engineering layouts were produced and capital costs were estimated based on calculated quantities and unit rates. Detailed analyses were also undertaken to assess the power capability and energy yields. At the Gold Creek, Devil Creek, Maclaren, Butte Creek, and Tyone sites, no detailed engineering or energy studies were undertaken; data from previous studies were used with capital cost estimates updated in 1980 levels. Approximate estimates of the potential average energy yield at the Butte Creek and Tyone sites were undertaken to assess the relative importance of these sites as energy producers.

The data presented in Table B.1 show that Devil Canyon, High Devil Canyon, and Watana are the most economic large energy producers in the basin. Sites such as Vee and Susitna III have only medium energy production, and slightly more costly that the previously mentioned damsites. Other sites such as Olson and Gold Creek are competitive provided they have additional upstream regulation. Sites such as Denali and Maclaren produce substantially higher cost energy than the other sites but can also be used to increase regulation of flow for downstream use.

(a) Site Screening

The objective of this screening process was to eliminate sites which would obviously not feature in the initial stages of the Susitna Basin development plan and which, therefore, did not deserve further study at this stage. Three basic screening criteria were used: environmental, alternative sites, and energy contribution.

The screening process involved eliminating all sites falling in the unacceptable environmental impact and alternative site categories. Those failing to meet the energy contribution criteria were also eliminated unless they had some potential for upstream regulation. The results of this process, described in detail in Reference 5, are as follows:

- The "unacceptable site" environmental category eliminated the Gold Creek, Olson, and Tyone sites.
- The alternative sites category eliminated the Devil Creek and Butte Creek sites.
- No additional sites were eliminated for failing to meet the energy contribution criteria. The remaining sites upstream from Vee, i.e., Maclaren and Denali, were retained to insure that further study be directed toward determining the need and viability of providing flow regulation in the headwaters of the Susitna.

(b) Engineering Layouts

In order to obtain a uniform and reliable data base for studying the seven sites remaining, it is necessary to develop engineering layouts and reevaluate the costs. In addition, staged developments at several of the larger dams were studied.

The basic objective of these layout studies was to establish a uniform and consistent development cost for each site. These layouts are consequently conceptual in nature and do not necessarily represent optimum project arrangements at the sites. Also, because of the lack of geotechnical information at several of the sites, judgmental decisions had to made on the appropriate foundation and abutment treatment. The accuracy of cost estimates made in these studies is of the order of plus or minus 30 percent.

(i) Design Assumptions

In order to maximize standardization of the layouts, a set of basic design assumptions was developed. These assumptions covered geotechnical, hydrologic, hydraulic, civil, mechanical, and electrical considerations and were used as guidelines to determine the type and size of the various components within the overall project layouts. As stated previously, other than at Watana, Devil Canyon, and Denali, little information regarding site conditions was available. Broad assumptions were made on the basis of the limited data, and those assumptions and the interpretation of data have been conservative.

It was assumed that the relative cost differences between rockfill and concrete dams at the site would either be marginal or greatly in favor of the rockfill. The more detailed studies carried out subsequently for the Watana and Devil Canyon sites support this assumption. Therefore, a rockfill dam has been assumed at all developments in order to eliminate cost discrepancies that might result from a consideration of dam-fill unit costs compared to concrete unit costs at alternative sites.

(ii) General Arrangements

A brief description of the general arrangements developed for the various sites is given below. Descriptions of Watana and Devil Canyon in this section are of the preliminary layouts and should not be confused with the proposed layouts in Exhibit A and Exhibit F. Figures B.6 to B.12 illustrate the layout details. Table B.2 summarizes the crest levels and dam heights considered.

In laying out the developments, conservative arrangements have been adopted, and whenever possible there has been a general standardization of the component structures.

- Devil Canyon (Figure B.6)

The development at Devil Canyon, located at the upper end of the canyon at its narrowest point, consists of a rockfill dam, single spillway, power facilities incorporating an underground powerhouse, and a tunnel diversion.

The rockfill dam would rise above the valley on the left abutment and terminate in an adjoining saddle dam of similar construction. The dam would be 675 feet above the lowest foundation level with a crest elevation of 1470 and a volume of 20 million cubic yards.

The spillway would be located on the right bank and would consist of a gated overflow structure and a concrete-lined chute linking the overflow structure with intermediate and terminal stilling basins. Sufficient spillway capacity would be provided to pass the Probable Maximum Flood safetly.

The power facilities would be located on the right abutment. The massive intake structure would be founded within the rock at the end of a deep approach channel and would consist of four integrated units, each serving individual tunnel penstocks. The powerhouse would house four 150-MW vertically mounted Francis type turbines driving overhead 165 MVA umbrella type generators.

As an alternative to the full power development in the first phase of construction, a staged powerhouse alternative was also investigated. The dam would be completed to its full height but with a initial plant installed capacity in 300-MW range. The complete powerhouse would be constucted together with penstocks and a tailrace tunnel for the initial two 150-MW units, together with concrete foundations for the future units.

- Watana (Figure B.7 and B.8)

For initial comparative study purposes,the dam at Watana is assumed to be a rockfill structure located on a similar alignment to that proposed in the previous COE studies. It would be similar in construction to the dam at Devil Canyon with an impervious core founded on sound bedrock and an outer shell composed of blasted rock excavated from a single quarry located on the left abutment. The dam would rise 880 feet from the lowest point on the foundation and have an overall volume of approximately 63 million cubic yards for a crest elevation of 2225.

The spillway would be located on the right bank and would be similar in concept to that at Devil Canyon with an intermediate and terminal stilling basin.

The power facilities located within the left abutment with similar intake, underground powerhouse, and water passage concepts to those at Devil Canyon would incorporate four 200-MW turbine/generator units giving a total output of 800-MW.

As an alternative to the initial full development at Watana, staging alternatives were investigated. These included staging of both dam and powerhouse construction. Staging of the powerhouse would be similar to that at Devil Canyon, with a Stage I installation of 400-MW and a further 400-MW in Stage II.

In order to study the alternative dam staging concept it was assumed that the dam would be constructed for a maximum operating water surface elevation some 200 feet lower than that in the final stage (see Figure B.8).

The powerhouse would be completely excavated to its final size during the first stage. Three oversized 135-MW units would be installed together with base concrete for an additional unit. A low level control structure and twin concrete-lined tunnels leading into a downstream stilling basin would form the first stage spillway.

For the second stage, the dam would be completed to its full height, the impervious core would be appropriately raised, and additional rockfill would be placed on the downstream face. It was assumed that before construction commences the top 400 feet of the first stage dam would be removed to ensure the complete integrity of the impervious core for the raised dam. A second spillway control structure would be constructed at a higher level and would incorporate a downstream chute leading to the Stage I spillway structure. The original spillway tunnels would be closed with concrete plugs. A new intake structure would be constructed utilizing existing gates and hoists, and new penstocks would be driven to connect with the existing ones. The existing intake would be sealed off. One additional 200 MW unit would be installed and the required additional penstock and tailrace tunnel constructed. The existing 135-MW units would be upgraded to 200 MW.

- High Devil Canyon (Figure B.9)

The development would be located between Devil Canyon and Watana. The 855 feet high rockfill dam would be similar in design to Devil Canyon, containing an estimated 48 million cubic yards of rockfill with a crest elevation of 1775. The left bank spillway and the right bank powerhouse facilities would also be similar in concept to Devil Canyon, with an installed capacity of 800-MW.

Two stages of 400-MW were envisaged in each which would be undertaken in the same manner as at Devil Canyon, with the dam initially constructed to its full height.

- Susitna III (Figure B.10)

The development would involve a rockfill dam with an impervious core approximately 670 feet high, a crest elevation of 2360, and a volume of approximately 55 million cubic yards. A concrete-lined spillway chute and a single stilling basin and would be located underground and the two diversion tunnels on the left bank.

- Vee (Figure B.11)

A 610 feet high rockfill dam founded on bedrock with a crest elevation of 2350 and total volume fo 10 million cubic yards was considered.

Since Vee is located further upstream than the other major sites the flood flows are correspondingly lower, thus allowing for a reduction in size of the spillway facilities. A spillway utilizing a gated overflow structure, chute, and flip bucket was adopted.

The power facilities would consist of a 400-MW underground powerhouse located in the left bank with a tailrace outlet well downstream of the main dam. A secondary rockfill dam would also be required in this vicinity to seal off a low point. Two diversion tunnels would be provided on the right bank.

- Maclaren (Figure B.12)

  The development would consist of a 185 feet high earthfill dam founded on pervious riverbed materials. The crest elevation of the dam would be 2405. This reservoir would essentially be used for reglating purposes. Diversion would occur through three conduits located in a open cut on the left bank and floods would be discharged via a side chute spillway and stilling basin on the right bank.

- Denali (Figure B.12)

  Denali is similar in concept to Maclaren. The dam would be 230 feet high, of earthfill construction, and would have a crest elevation of 2555. As for Maclaren, no generating capacity would be included. A combined diversion and spillway facility would be provided by twin concrete conduits founded in open cut excavation in the right bank and discharging into a common stilling basin.

(c) Capital Costs

For purposes of initial comparisons of alternatives, construction quantities were determined for items comprising the major works and structures at the site. Where detail or data were not sufficient for certain work, quantity estimates were made on the basis of previous Acres' experience and the general knowledge of site conditions reported in the literature. In order to determine total capital costs for various structures, unit costs have been developed for the items measured. These have been estimated on the basis of review of rates used in previous studies, and of rates used on similar works in Alaska and elsewhere. Where applicable, adjustment factors based on geography, climate, manpower and accessibility were used. Technical publications have also been reviewed for basic rates and escalation factors.

The total capital costs developed are shown in Table B.1 and B.2. It should be noted that the capital costs for Maclaren and Denali shown in Table B.1 have been adjusted to incorporate the costs of generation plants with capacities of 55-MW and 60-MW, respectively. Additional data on the projects are summarized in Table B.3.

1.4 - Formulation of Susitna Basin Development Plans

The results of the site screening process described above indicate that the Susitna Basin development plan should incorporate a combination of several major dams and powerhouses located at one or more of the following sites:

- Devil Canyon;
- High Devil Canyon;
- Watana;
- Susitna III; or
- Vee.

Supplementary upstream flow regulation could be provided by structures at:

- Maclaren; and
- Denali.

Cost estimates of these projects are itemized on Table B.4.

A computer assisted screening process identified the plans that are most economic as those of Devil Canyon/Watana or High Devil Canyon/Vee. In addition to these two basic development plans, a tunnel scheme which provides potential environmental advantages by replacing the Devil Canyon dam with a long power tunnel and a development plan involving Watana Dam was also introduced.

The criteria used at this stage of the process for selection of preferred Susitna Basin development plans are mainly economic (see Figure B.3). Environmental considerations are incorporated into the further assessment of the plans finally selected.

The results of the screening process are shown in Table B.5. Because of the simplifying assumptions that were made in the screening model, the three best solutions from an economic point of view are included in the table.

The most important conclusions that can be drawn are as follows:

- For energy requirements of up to 1,150 Gwh, the High Devil Canyon, Devil Canyon or the Watana sites individually provided the most economic energy. The difference between the costs shown on Table B.4 is around 10 percent, which is similar to the accuracy that can be expected from the screening model.

- For energy requirements of between 1,750 and 3,500 Gwh, the High Devil Canyon site is the most economic.

- For energy requirements of between 3,500 and 5,250 Gwh the combinations of either Watana and Devil Canyon or High Devil Canyon and Vee are most economic.

- The total energy production capability of the Watana/Devil Canyon developments is considerably larger than that of the High Devil Canyon/Vee alternative and is the only plan capable of meeting energy demands in the 6,000 Gwh range.

(a) Tunnel Alternative

A scheme involving a long power tunnel could conceivably be used to replace the Devil Canyon dam is the Watana/Devil Canyon development plan. It could develop similar head for power generation and may provide some environmental advantages by avoiding inundation of Devil Canyon. Obviously, because of the low winter flows in the river, a tunnel alternative could be considered only as a second stage to the Watana development.

Conceptually, the tunnel alternatives would comprise the following major components in some combination, in addition to the Watana dam reservoir and associated powerhouse:

- Power tunnel intake works;

- One or two power tunnels of up to forty feet in diameter and up to thirty miles in length;

- A surface or underground powerhouse with a capacity of up to 1200 MW;

- A re-regulation dam if the intake works are located downstream from Watana; and

- Arrangements for compensation flow in the bypassed river reach.

Four basic alternative schemes were developed and studied. Figure B.13 is a schematic illustration of these schemes. All schemes assumed an initial Watana development with full reservoir supply level at Elevation 2200 and the associated powerhouse with an installed capacity of 800 MW. Table B.6 lists all the pertinent technical information. Table B.7 lists the power and energy yields for the four schemes. Table B.8 itemizes the capital cost estimate.

Based on the foregoing economic information, Scheme 3 (Figures B.14 and B.15) produces the lowest cost energy by a factor of nearly 2.

A review of the environmental impacts associated with the four tunnel schemes indicates that Scheme 3 would have the least impact, primarily because it offers the best opportunities for regulating daily flows downstream from the project. Based on this assessment, and because of its almost 2 to 1 economic advantage, Scheme 3 was selected as the only scheme worth further study (see Development Selection Report for detailed analysis). The capital cost estimate for Scheme 3 appears in Table B.8. The estimates also incorporate single and double tunnel options. For purposes of these studies, the double tunnel option has been selected because of its superior reliability. It should also be recognized that the cost estimates associated with the tunnels are probably subject to more variation than those associated with the dam schemes due to geotechnical uncertainties. In an attempt to compensate for these uncertainties, economic sensitivity analyses using both higher and lower tunnel costs have been conducted.

(b) Additional Basin Development Plan

As noted, the Watana and High Devil Canyon dam sites appear to be individually superior in economic terms to all others. An additional plan was therefore developed to assess the potential for developing these two sites together. For this scheme, the Watana dam would be developed to its full potential. The High Devil Canyon dam would be constructed to a crest elevation of 1470 feet to fully utilize the head downstream from Watana.

(c) Selected Basin Development Plans

The essential objectives of this step in the development selection process is defined as the identification of those plans which appear to warrant further, more detailed evaluation. The results of final screening process indicate that the Watana/Devil Canyon and the High Devil Canyon/Vee plans are clearly superior to all other dam combinations. In addition, it was decided to study further tunnel Scheme 3 as an alternative to the High Devil Canyon dam and a plan combining a Watana/High Devil Canyon.

Associated with each of these plans are several options for staged development. For this more detailed analysis of these basic plans, a range of different approaches to staging the developments was considered. In order to keep the total options to a reasonable number and also to maintain reasonably large staging steps consistent with the total development size, staging of only the two larger developments, i.e., Watana and High Devil Canyon, was considered. The basic staging concepts adopted for these developments involved staging both dam and powerhouse construction, or alternatively just staging powerhouse construction. Powerhouse stages were considered in 400 MW increments.

Four basic plans and associated subplans are briefly described below. Plan 1 involves the Watana-Devil Canyon sites, Plan 2 the High Devil Canyon-Vee sites, Plan 3 the Watana-tunnel concept, and Plan 4 the Watana-High Devil Canyon sites. Under each plan several alternative subplans were identified, each involving a different staging concept. Summaries of these plans are given in Table B.9.

(i) Plan 1

- Subplan 1.1: The first stage involves constructing Watana dam to its full height and installing 800 MW. Stage 2 involves constructing Devil Canyon dam and installing 600 MW.

- Subplan 1.2: For this Subplan, construction of the Watana dam is staged from a crest elevation of 2060 feet to 2225 feet. The powerhouse is also staged from 400 MW to 800 MW. As for Subplan 1.1, the final stage involves Devil Canyon with an installed capacity of 600 MW.

- Subplan 1.3: This Subplan is similar to Subplan 1.2 except that only the powerhouse and not the dam at Watana is staged.

(ii) Plan 2

- Subplan 2.1: This Subplan involves constructing the High Devil Canyon dam first with an installed capacity of 800 MW. The second stage involves constructing the Vee dam with an installed capacity of 400 MW.

- Subplan 2.2: For this Subplan, the construction of High Devil Canyon is staged from a crest elevation of 1630 to 1775 feet. The installed capacity is also staged from 400 to 800 MW. As for Subplan 2.1, Vee follows with 400 MW of installed capacity.

- Subplan 2.3: This Subplan is similar to Subplan 2.2 except that only the powerhouse and not the dam at High Devil Canyon is staged.

(iii) Plan 3

- Subplan 3.1: This Subplan involves initial construction of Watana and installation of 800 MW capacity. The next stage involves the construction of the downstream re-regulation dam to a crest elevation of 1500 feet and a 15 mile long tunnel. A total of 300 MW would be installed at the end of the tunnel and a further 30 MW at the re-regulation dam. An additional 50 MW of capacity would be installed at the Watana powerhouse to facilitate peaking operations.

- Subplan 3.2: This Subplan is essentially the same as Subplan 3.1 except that construction of the initial 800 MW powerhouse at Watana is staged.

(iv) Plan 4

This single plan was developed to evaluate the development of the two most economic dam sites, Watana and High Devil Canyon, jointly. Stage 1 involves constructing Watana to its full height with an installed capacity of 400 MW. Stage 2 involves increasing the capacity at Watana to 800 MW. Stage 3 involves constructing High Deveil Canyon to a crest elevation of 1470 feet so that the reservoir extends to just downstream of Watana. In order to develop the full head between Watana and Portage Creek, an additional smaller dam is added downstream of High Devil Canyon. This dam would be located just upstream from Portage Creek so as not to interfere with the anadromous fisheries and would have a crest elevation of 1030 feet and an installed capacity of 150 MW. For purposes of these studies, this site is referred to as the Portage Creek site.

1.5 - Evaluation of Basin Development Plan

The overall objective of this step in the evaluation process was to select the preferred basin development plan. A preliminary evaluation of plans was initially undertaken to determine broad comparisons of the available alternatives. This was followed by appropriate adjustments to the plans and a more detailed evaluation and comparison.

In the process of initially evaluating the final four schemes, it became apparent that there would be environmental problems associated with allowing daily peaking operations from the most downstream reservoir in each of the plans described above. In order to avoid these potential problems while still maintaining operational flexibility to peak on a daily basis, re-regulation facilities were incorporated in the four basic plans. These facilities incorporate both structural measures such as re-regulation dams and modified operational procedures. Details of these modified plans, referred to as E1 to E4, are listed in Table B.10.

The plans listed in Table B.10 were subjected to a more detailed analysis as described in the following section.

(a) Evaluation Methodology

The approach to evaluating the various basin development plans described above is twofold:

- For determining the optimum staging concept associated with each basic plan (i.e., the optimum subplan), only economic criteria are used and the least cost staging concept is adopted.

- For assessing which plan is the most appropriate, a more detailed evaluation process incorporating economic, environmental, social and energy contribution aspects is taken into account.

Economic evaluation of any Susitna Basin development plan requires that the impact of the plan on the cost of energy to the Railbelt area consumer be assessed on a systemwide basis. Since the consumer is supplied by a large number of different generating sources, it is necessary to determine the total Railbelt system cost in each case to compare the various Susitna Basin development options.

The primary tool used for system costs was the mathematical model developed by the Electricity Utility Systems Engineering Department of the General Electric Company. The model is commonly known as OGP5 or Optimized Generation Planning Model, Version 5. The following information is paraphrased from GE literature on the program.

The OGP5 program was developed over ten years to combine the three main elements of generation expansion planning (system reliability, operating and investment costs) and automate generation addition decision analysis. OGP5 will automatically develop optimum generation expansion patterns in terms of economics, reliability and operation. Many utilities use OGP5 to study load management, unit size, capital and fuel costs, energy storage, forced outage rates, and forecast uncertainty.

The OGP5 program requires an extensive system of specific data to perform its planning function. In developing an optimal plan, the program considers the existing and committed units (planned and under construction) available to the system and the characteristics of these units including age, heat rate, size and outage rates as the base generation plan. The program then considers the given load forecast and operation criteria to determine the need for additional system capacity based on given reliability criteria. This determines "how much" capacity to add and "when" it should be installed. If a need exists during any monthly iteration, the program will consider additions from a list of alternatives and select the available unit best fitting the system needs. Unit selection is made by computing production costs for the system for each alternative included and comparing the results.

The unit resulting in the lowest system production cost is selected and added to the system. Finally, an investment cost analysis of the capital costs is completed to answer the question of "what kind" of generation to add to the system.

The model is then further used to compare alternative plans for meeting variable electrical demands, based on system reliability and production costs for the study period.

A minor limitation inherent in the use of the OGP5 model is that the number of years of simulation is limited to 20. To overcome this, the study period of 1980 to 2040 has been broken into three separate segments for study purposes. These segments are common to all system generation plans.

The first segment has been assumed to be from 1980 to 1990. The model of this time period included all committed generation units and is assumed to be common to all generation scenarios.

The end point of this model becomes the beginning of each 1990-2010 model.

The model of the first two time periods considered (1980 to 1990, and 1990 to 2010) provides the total production costs on a year-to-year basis. These total costs include, for the period of modeling, all costs of fuel and operation and maintenance of all generating units included as part of the system. In addition, the completed production costs includes the annualized investment costs of any production plans added during the period of study. A number of factors which contribute to the ultimate cost of power to the consumer, are not included in this model. These are common to all scenarios and include:

- All investment costs to plants in service prior to 1981;

- Costs of transmission systems in service both at the transmission and distribution level; and

- Administrative costs of utilities for providing electric service to the public.

Thus, it should be recognized that the production costs modeled represent only a portion of ultimate consumer costs and in effect are only a portion, albeit major, of total costs.

The third period, 2010 to 2040, was modeled by assuming that production costs of 2010 would recur for the additional 30 years to 2040. This assumption is believed to be reasonable given the limitations on forecasting energy and load requirements for this period. The additional period to 2040 is required to at least take into account the benefit derived or value of the addition of hydroelectric power plant which has a useful life of fifty years or more.

The selection of the preferred generation plan is based on numerous factors. One of these is the cost of the generation plan. To provide a consistent means of assessing the production cost of a given generation scenario, each production cost total has been converted to a 1980 present worth basis. The present worth cost of any generation scenario is made up of three cost amounts. The first is present worth cost (PWC) of the first ten years of study (1981 to 1990), the second is the PWC of the scenario assumed during 1990 to 2010 and the third the PWC of the scenario in 2010 assumed to recur for the period 2010 to 2040. In this way the long-term (60 years) PWC of each generation scenario in 1980 dollars can be compared.

A summary of the input data to the model and a discussion of the results follow.

(i) Initial Economic Analyses

Table B.11 lists the results of the first series of economic analyses undertaken for the basic Susitna Basin development plans listed in Table B.10. The information provided includes the specified on-line dates for the various stages of the plans, the OGP5 run index number, the total installed capacity at year 2010 by category, and the total system present-worth cost in 1980 for the period 1980 to 2040. Matching of the Susitna development to the load growth for Plans E1, E2, and E3 is shown in Figure B.16, B.17 and B.18 respectively. After 2010, steady state conditions are assumed and the then-existing generation mix and annual costs for 2010 are applied to the years 2011 to 2040. This extended period of time is necessary to ensure that the hydroelectric options being studied, many of which only come on-line around 2000, are simulated as operating for periods approaching their economic lives and that their full impact on the cost of the generation system is taken into account.

- Plan E1 - Watana/Devil Canyon

. Staging the dam at Watana (Plan E1.2) is not as economic as constructing it to its full height (Plan E1.1 and E1.3). The present worth advantage of not staging the dam amounts to $180 million in 1980 dollars.

The results indicate that, with the level of analysis performed, there is no discernible benefit in staging construction of the Watana powerhouse (Plan E1.1 and E1.3). However, Plan E1.4 results indicates that, should the powerhouse size at Watana be restricted to 400 MW, the overall system present worth would increase.

Additional runs performed for variations of Plan E1.3 indicated that system present worth would increase by $1,110 million if the Devil Canyon dam was not constructed. A five year delay in construction of the Watana dam would increase system present worth by $220 million.

- Plan E2 - High Devil Canyon/Vee

  . The results for Plan E2.3 indicate that the system present worth is $520 million more than Plan E1.3. Present worth increases also occur if the Vee dam stage is not constructed. A reduction in present worth of approximately $160 million is possible if the Chakachamna hydroelectric project is constructed instead of the Vee dam.

  . The results of Plan E2.1 indicate that total system present worth would increase by $250 million if the total capacity at High Devil Canyon were limited to 400 MW.

- Plan E3 - Watana/Tunnel

  The results for Plan E3.1 illustrate that the tunnel scheme versus the Devil Canyon dam scheme (E1.3) adds approximately $680 million to the total system present worth cost. The availability of reliable geotechnical data would undoubtedly have improved the accuracy of the cost estimates for the tunnel alternative. For this reason, a sensitivity analysis was made as a check to determine the effect of halving the tunnel costs. This analysis indicates that the tunnel scheme is still more costly than constructing the Devil Canyon dam.

- Plan E4 - Watana/High Devil Canyon/Portage Creek

  The results indicate that system present worth associated with Plan E4.1, excluding the Portage Creek site development, are $200 million more than the equivalent E1.3 plan. If the Portage Creek development is included, the present worth difference would be even greater.

(ii) Load Forecast Sensitivity Analyses

The plans with the lowest present-worth cost were subjected to further sensitivity analyses to assess the economic impacts of various load growths. These results are summarized in Table B.12.

The results for low load forecasts illustrate that the most viable Susitna Basin development plan is the Watana-Devil Canyon plan with a capacity of 800 MW, which has a present worth cost of $210 million less than its closest competitor, the High Devil Canyon-Vee plan.

For the high load forecasts, the results indicate that the Plan E1.3 has a present worth cost of $1040 million less than E2.3.

(b) Evaluation Criteria

The following criteria were used to evaluate the shortlisted basin development plans. These criteria generally contain the requirements of the generic process with the exception that an additional criterion, energy contribution, is added in order to ensure that full consideration is given to the total basin energy potential developed by the various plans.

(i) Economic

Plans were compared using long-term present worth costs, calculated using the OGP5 generation planning model. The parameters used in calculating the total present-worth cost of the total Railbelt generating system for the period 1980 to 2040 are listed in Table B.13 and B.14. Load forecasts used in the analysis are presented in Section 5.1(b).

(ii) Environmental

A qualitative assessment of the environmental impact on the ecological, cultural, and aesthetic resources is undertaken for each plan. Emphasis is placed on identifying major concerns so that these could be combined with the other evaluation attributes in an overall assessment of the plan.

(iii) Social

This attribute includes determination of the potential nonrenewable resource displacement, the impact on the state and local economy, and the risks and consequences of major structural failures due to seismic events. Impacts on the economy refer to the effects of an investment plan on economic variables.

(iv) Energy Contribution

The parameter used is the total amount of energy produced from the specific development plan. An assessment of the energy development foregone is also undertaken. The energy loss that is inherent to the plan and cannot easily be recovered by subsequent staged developments is of greatest concern.

(c) Results of Evaluation Process

The various attributes outlined above have been determined for each plan and are summarized in Tables B.15 through B.23. Some of the attributes are quantitative while others are qualitative. Overall evaluation is based on a comparison of similar types of attributes for each plan. In cases where the attributes associated with one plan all indicate equality or superiority with respect to another plan, the decision as to the best plan is clear cut. In other cases where some attributes indicate superiority and others inferiority, differences are highlighted and trade-off decisions are made to determine the preferred development plan. In cases where these trade-offs have had to be made, they were relatively straightforward, and the decision-making process can, therefore, be regarded as effective and consistent. In addition, these trade-offs are clearly identified so the recorder can independently assess the judgment decisions made.

The overall evaluation process is conducted in a series of steps. At each step, only two plans are compared. The superior plan is then taken to the next step for evaluation against a third plan.

(i) Devil Canyon Dam Versus Tunnel

The first step in the process involves the comparison of the Watana-Devil Canyon dam plan (E1.3) and the Watana-Tunnel plan (E3.1). Since Watana is common to both plans, the evaluation is based on a comparison of the Devil Canyon dam and Scheme 3 tunnel alternative.

In order to assist in the evaluation in terms of economic criteria, additional information obtained by analyzing the results of the OGP5 computer runs is shown in Table B.15. This information illustrates the breakdown of the total system present worth cost in terms of capital investment, fuel, and operation and maintenance costs.

- Economic Comparison

From an economic point of view, the Watana-Devil Canyon dam scheme is superior. As summarized in Tables B.15 and B.16, on a present worth basis the tunnel scheme is $680 million more expensive than the dam scheme. For a low demand growth rate, this cost difference would be reduced slightly to $650 million. Even if the tunnel scheme costs are halved, the total cost difference would still amount to $380 million. As highlighted in Table B.16 consideration of the sensitivity of the basic econopmic evaluation to potential changes in capital cost estimate, the period of economic analysis, the discount rate, fuel costs, fuel cost escalation, and economic plant life do not change the basic economic superiority of the dam scheme over the tunnel scheme.

- Environmental Comparison

The environmental comparison of the two schemes is summarized in Table B.17. Overall, the tunnel scheme is judged to be superior because:

- It offers the potential for enhancing anadromous fish populations downstream of the re-regulation dam due to the more uniform flow distribution that will be achieved in this reach;

- It would inundate 13 miles less of resident fisheries habitat in river and major tributaries;

- It has a lower potential for inundating archeological sites due to smaller reservoir involved; and

- It would preserve much of the characteristics of the Devil Canyon gorge which is considered to be an aesthetic and recreational resource.

- Social Comparison

Table B.18 summarizes the evaluation in terms of the social criteria of the two schemes. In terms of impact on state and local economics and risks because of seismic exposure, the two schemes are rated equal. However, the dam scheme has, due to its higher energy yield, more potential for displacing nonrenewable energy resources, and therefore has a slight overall advantage in terms of the social evaluation criteria.

- Energy Comparison

Table B.19 summarizes the evaluation in terms of the energy contribution criteria. The results shown that the dam scheme has a greater potential for energy production and develops a larger portion of the basin's potential. The dam scheme is therefore judged to be superior from the energy contribution standpoint.

- Overall Comparison

The overall evaluation of the two schemes is summarized in Table B.20. The estimated cost saving of $680 million in favor of the dam scheme plus the additional energy produced are considered to outweigh the reduction in the overall environmental impact of the tunnel scheme. The dam scheme is therefore judged to be superior overall.

(ii) Watana-Devil Canyon Versus High Devil Canyon-Vee

The second step in the development selection process involves an evaluation of the Watana-Devil Canyon (E1.3) and the High Devil Canyon-Vee (E2.3) development plans.

- Economic Comparison

In terms of the economic criteria (see Table B.15 and B.16) the Watana-Devil Canyon plan is less costly by $520 million. Consideration of the sensitivity of this decision to potential changes in the various parameters considered (i.e., load forecast, discounted rates, etc.) does not change the basic superiority of the Watana-Devil Canyon Plan.

- Environmental Comparison

The evaluation in terms of the environmental criteria is summarized in Table B.21. In assessing these plans, a reach-by-reach comparison was made for the section of the Susitna River between Portage Creek and the Tyone River. The Watana-Devil Canyon scheme would create more potential environmental impacts in the Watana Creek area. However, it is judged that the potential environmental impacts which would occur above the Vee Canyon dam with a High Devil Canyon-Vee development are more severe in overall comparison.

Of the seven environmental factors considered in Table B.17, except for the increased loss of river valley, bird and black bear habitat the Watana-Devil Canyon development plan is judged to be more environmentally acceptable than the High Canyon-Vee plan.

- Energy Comparison

The evaluation of the two plans in terms of energy contribution criteria is summarized in Table B.22. The Watana-Devil Canyon scheme is assessed to be superior because of its higher energy potential and the fact that it develops a higher proportion of the basin's energy potential.

- Social Comparison

Table B.18 summarizes the evaluation in terms of the social criteria. As in the case of the dam versus tunnel comparison, the Watana-Devil Canyon plan is judged to have a slight advantage over the High Devil Canyon-Vee plan. This is because of its greater potential for displacing nonrenewable resources.

1.6 - Preferred Susitna Basin Development Plan

One-on-one comparisons of the Watana-Devil Canyon plan with the Watana-tunnel plan and the High Devil Canyon-Vee plans are judged to favor the Watana-Devil Canyon plan in each case.

The Watana-Devil Canyon plan was therefore selected as the preferred Susitna Basin development plan, and the basis for continuation of more detailed design optimization and environmental studies.

2 - ALTERNATIVE FACILITY DESIGNS, PROCESSES AND OPERATIONS

2.1 - Susitna Hydroelectric Development

As originally conceived the Watana project initially comprised an earthfill dam, with a crest elevation of 2225 and 400 MW of generating capacity scheduled to commence operation in 1993. An additional 400 MW would be brought on-line in 1996. At Devil Canyon an additional 400 MW would be installed to commence operation in the year 2000. Detailed studies of each project have led to refinement and optimization of designs in terms of a number of key factors, including updated load forecasts and economics. Geotechnical and environmental constraints identified as a result of continuing field work have also greatly influenced the currently recommended design concepts.

Plan formulation and alternative facility designs considered for the Watana and Devil Canyon developments are discussed in this section.

This section includes the alternatives studied and the reason for selecting the proposed plan. Background information on the site characteristics as well as additional detail on the plan formulation process are included in the Design Report of Exhibit F and the referenced reports.

2.2 - Watana Project Formulation

This section describes the evolution of the general arrangement of the Watana project which, together with the Devil Canyon project, comprises the development plan proposed. The process by which reservoir operating levels and the installed generating capacity of the power facilities were established is presented, together with the means of handling floods expected during construction and subsequent project operation.

The main components of the Watana development are as follows:

- Main dam;
- Diversion facilities;
- Spillway facilities;
- Outlet facilities;
- Emergency release facilities; and
- Power facilities.

A number of alternatives are available for each of these components and they can be combined in a number of ways. The following paragraphs describe the various components and methodology for the preliminary, intermediate, and final screening and review of alternative general arrangement of the components, together with a brief description of the selected scheme. This section presents the alternative arrangements studied for the Watana project.

(a) Selection of Reservoir Levels

The selected elevation of the Watana dam crest is based on considerations of the value of the hydroelectric energy produced from the associated reservoir, geotechnical constraints on reservoir levels, and freeboard requirements. Firm energy, average annual energy, construction costs, and operation and maintenance costs were determined for the Watana development with dam crest elevations of 2240, 2190, and 2140. The relative value of energy produced in terms of the present worth of the long-term production costs (LTPW) for each of these three dam elevations was determined by means of the OGP5 generation planning model described in Section 1 of this Exhibit. The physical constraints imposed on dam height and reservoir elevation by geotechnical considerations were reviewed and incorporated into the crest elevation selection process. Finally, freeboard requirements for the PMF and settlement of the dam after construction or as a result of seismic activity were taken into account.

(i) Methodology

Firm and average annual energy produced by the Susitna development are based on 32 years of hydrological records. The energy produced was determined by using a multi-reservoir simulation of the operation of the Watana and Devil Canyon reservoirs. A variety of reservoir drawdowns were examined, and drawdowns producing the maximum firm energy consistent with engineering feasibility and cost of the intake structure were selected. Minimum flow requirements were established at both project sites based on downstream fisheries considerations.

To meet system demand the required maximum generating capability at Watana in the period 1993 and 2010 ranges from 665 MW to 908 MW. For the reservoir level determinations, energy estimates were made on the basis of assumed average annual capacity requirements of 680 MW at Watana in 1993, increasing to 1020 MW at Watana in 2007, with an additional 600 MW at Devil Canyon coming online in the year 2002. The long term present worth costs of the generation system required to meet the Railbelt energy demand were then determined for each of the three crest elevations of the Watana dam using the OGP5 model.

The construction cost estimates used in the OGP5 modeling process for the Watana and Devil Canyon projects were based on preliminary conceptual layouts and construction schedules. Further refinement of these layouts has taken place during the optimization process. These refinements have no significant impact on the reservoir level selection.

(ii) Economic Optimization

Economic optimization of the Watana reservoir level was based on an evaluation of three dam crest elevations of 2240, 2190, and 2140. These crest elevations apply to the central portion of the embankment with appropriate allowances for freeboard and seismic settlement, and correspond to maximum operating levels of the reservoir of 2215, 2165, and 2115 feet, respectively. Average annual energy calculated for each case using the reservoir simulation model are given in Table B.24, together with corresponding project construction costs.

In the determination of LTPW, the Susitna capital costs were adjusted to include an allowance for interest during construction and then used as input to the OGP5 model. Simulated annual energy yields were distributed on a monthly basis by the reservoir operation model to match as closely as possible the projected monthly energy demand of the Railbelt and then input to the OGP5 model. The LTPW of meeting the Railbelt energy demand using the Susitna development as the primary source of energy was then determined for each of the three reservoir levels.

The results of these evaluations are shown in Table B.25, and plots showing the variation of the LTPW with dam crest elevation are shown in Figure B.19. This figure indicates that on the basis of the assumptions used, the minimum LTPW occurs at a Watana crest elevation ranging from approximately 2160 to 2200 (reservoir levels 2140 to 2180 feet). A higher dam crest will still result in a development which has an overall net economic benefit relative to thermal energy sources. However, it is also clear that as the height of the Watana dam is increased, the unit costs of additional energy produced at Watana is somewhat greater than for the displaced thermal energy source. Hence, the LTPW of the overall system would increase. Conversely, as the height of the dam is lowered, and thus Watana produces less energy, the unit cost of the energy produced by a thermal generation source to replace the lost Susitna energy is more expensive than Susitna energy. In this case also, the LTPW increases.

(iii) Geotechnical Considerations

On the north side of the reservoir created by the Watana dam a relict channel of considerable depth connects the reservoir to Tsusena Creek. As the water surface elevation of the reservoir is increased up to and beyond 2200 feet, a low area in the relict channel would require costly water retaining structures to be built and other measures to be taken. In addition to the cost the technical feasibility of these measures is not as certain as desired on a project of this magnitude. Because of the considerations relating to seismic stability, seepage problems and permafrost conditions in the relict channel area, the hydraulic head at the upstream end of the relict channel should be limited wherever possible. By comparing normal reservoir levels plus flood surcharge to ground surface contours, it was determined that with normal reservoir levels of 2185 and a small freeboard dike the following conditions would exist:

- For flood magnitudes up to the 1:10,000-year event, there would be no danger of overtopping the lowest point in the relict channel.

- For the PMF a freeboard dike in the low area of up to 10 feet in height would provide adequate protection. This dike would be wetted only a few days during a PMF event.

- If seismic settlement or settlement due to permafrost melting did occur, the combination of the 10 feet freeboard dike constructed on a suitable foundation plus normal reservoir level of 2185 feet would ensure that breakthrough in the relict channel area would not occur.

With this approach, the Watana project will develop the maximum energy reasonably available without incurring the need for costly water retaining structures in the relict channel area.

(iv) Conclusions

It is important to establish clearly the overall objective used as a basis for setting the Watana reservoir level. An objective which would minimize the LTPW energy cost would lead to selection of a slightly lower reservoir level than an objective which would maximize the amount of energy which can be obtained from the available resource, while doing so with a technically sound project.

The three values of LTPW developed by the OGP5 computer runs defined a relationship between LTPW and Watana dam height which is relatively insensitive to dam height. This is highlighted by the curve of LTPW versus dam height in Figure B.19. This figure shows there is only a slight variation in the LTPW for the range of dam heights included in the analysis. Thus, from an economic standpoint the optimum crest elevation could be considered as varying over a range of elevations from 2140 to 2220 with little effect on project economics. The main factors in establishing the upper limit of dam height were consequently the geotechnical considerations discussed in (c) above.

The normal maximum operating level of the reservoir was therefore set at Elevation 2185, allowing the objective of maximizing the economic use of the Susitna resource still to be satisfied.

(b) Selection of Installed Capacity

The generating capacity to be installed at both Watana and Devil Canyon was determined on the basis of generation planning studies described in Sections 6 and 8 of Reference 4 together with appropriate consideration of the following:

- Available firm and average energy from Watana and Devil Canyon;
- The forecast energy demand and peak load demand of the system;
- Available firm and average energy from other existing and committed plant;
- Capital cost and annual operating costs for Watana and Devil Canyon;
- Capital cost and annual operating costs for alternative sources of energy and capacity;
- Environmental constraints on reservoir operation; and
- Turbine and generator operating characteristics.

(i) Methodology

The following procedure was used to select the installed capacity at Watana:

- The firm and average energy available at both Watana and Devil Canyon was determined using a reservoir simulation program.

- A determination was then made of the generating capacity required to utilize the available energy from the Susitna Project in the hydrological years of record, based on the following assumptions:

- In a wet year, energy developed at either Watana or Devil Canyon displaces excess thermal energy (from coal, gas turbine, combined cycle, or diesel plants).

- In an average year where thermal energy is required to meet system energy demand, hydro energy is used either to satisfy peak demand with thermal energy supplying base load (Option 1) or to supply base load requirements with thermal energy at peak demand (Option 2). The actual choice is based on dispatching the most economic energy first.

- Devil Canyon energy is used predominantly as base load energy because of environmental constraints on downstream flow variations.

- The maximum installed capacity was determined on the basis of the established peak generating capacity required plus any hydro standby or spinning reserve equipment.

(ii) Watana Installed Capacity

The required total capacity at Watana in a wet year, excluding standby and spinning reserve capacity, is summarized below. The capacities are based on the medium load forecast.

	Capacity (MW)					
	Option 1			Option 2		
Demand Year	Watana Peak	Thermal Base	Devil Canyon Base	Watana Base	Thermal Peak	Devil Canyon Base
1993	801	0	0	801	0	0
1995	839	0	0	839	0	0
2000	874	66	0	742	198	0
2002 (Including Devil Canyon)	660	0	354	660	0	354
2005 (Including Devil Canyon)	750	0	376	750	0	376
2010 (Including Devil Canyon)	900	0	493	900	0	493

On the basis of this evaluation, the ultimate power generation capability at Watana was selected as 1020 MW for design purposes to allow a margin for hydro spinning reserve and standby for forced outage. This installation also provides a small margin in the event that the load growth exceeds the medium load forecast.

(iii) Unit Capacity

Selection of the unit size for a given total capacity is a compromise between the initial least cost solution, generally involving a scheme with a smaller number of large capacity units, and the improved plant efficiency and security of operation provided by a larger number of smaller capacity units. Other factors include the size of each unit as a proportion of the total system load and the minimum anticipated load on the station. Any requirement for a minimum downstream flow would also affect the selection. Growth of the actual load demand is also a significant factor, since the installation of units may be phased to match the actual load growth. The number of units and their individual ratings were determined by the need to deliver the required peak capacity in the peak demand month of December at the minimum December reservoir level with the turbine wicket gates fully open.

An examination was made of the economic impact on power plant production costs of various combinations of a number of units and rated capacity which would provide the selected total capacity of 1020 MW. For any given installed capacity, plant efficiency increases as the number of units increases. The assumed capitalized value used in this evaluation was $1.00 per average annual kWh over project life, based on the economic analysis completed for the thermal generation system. Variations in the number of units and capacity will affect the cost of the power intakes, penstocks, powerhouse, and tailrace. The differences in these capital costs were estimated and included in the evaluation. The results of this analysis are presented below.

Number of Units	Rated Capacity of Unit (MW)	Capitalized Value of Additional Energy ($ Millions)	Additional Capital Cost ($ Millions)	Net Benefit ($ Millions)
4	250	0	0	0
6	170	40	31	9
8	125	50	58	-8

It is apparent from this analysis that a six-unit scheme with a net benefit of approximately $9 million is the most economic alternative. This scheme also offers a higher degree of flexibility and security of operation compared to the four-unit alternative, as well as advantages if unit installation is phased to match actual load growth. The net economic benefit of the six unit scheme is $17 million greater than that of the eight-unit scheme, while at the same time no significant operational or scheduling advantages are associated with the eight-unit scheme.

A scheme incorporating six units each with a rated capacity of 170 MW, for a total of 1020 MW, has been adopted for all Watana alternatives.

(c) Selection of the Spillway Design Flood

Normal design practice for projects of this magnitude, together with applicable design regulations, require that the project be capable of passing the Probable Maximum Flood (PMF) routed through the reservoir without endangering the dam.

In addition to this requirement, the project should have sufficient spillway capacity to safely pass a major flood of lesser magnitude than the PMF without damaging the main dam or ancillary structures. The frequency of occurrence of this flood, known as the spillway design flood or Standard Project Flood (SPF), is generally selected on the basis of an evaluation of the risks to the project if the spillway design flood is exceeded, compared to the costs of the structures required to safely discharge the flood. For this study, a spillway design flood with a return frequency of 1:10,000 years was selected for Watana. A list of spillway design flood frequencies and magnitudes for several major projects is presented below.

Project	Spillway Design Flood		Basin PMF (cfs)	Spillway Capacity After Routing (cfs)*
	Frequency	Peak Inflow (cfs)		
Mica, Canada	PMF	250,000	250,000	150,000
Churchill Falls, Canada	1:10,000	600,000	1,000,000	230,000
New Bullards, USA	PMF	226,000	226,000	170,000
Oroville, USA	1:10,000	440,500	711,400	440,500
Guri, Venezuela (final stage)	PMF	1,000,000	1,000,000	1,000,000
Itaipu, Brazil	PMF	2,195,000	2,195,000	2,105,000
Sayano, USSR	1:10,000	480,000	N/A	680,000

*All spillways except Sayano have capacity to pass PMF with surcharge.

The flood frequency analysis produced the following values:

Flood	Frequency	Inflow Peak
Probable Maximum	--	326,000 cfs
Spillway Design	1:10,000 years	156,000 cfs

Additional capacity required to pass the PMF will be provided by an emergency spillway consisting of a fuse plug and rock channel on the right bank.

(d) Main Dam Alternatives

This section describes the alternative types of dams considered at the Watana site and the basis for the selected alternative.

(i) Comparison of Embankment and Concrete Type Dams

The selection between an embankment type or a concrete type dam is usually based on the configuration of the valley, the condition of the foundation rock, depth of the overburden, and the relative availability of construction materials. Previous studies by the COE envisaged an embankment dam at Watana. Initial studies completed as part of this current evaluation included comparison of an earthfill dam with a concrete arch dam at the Watana site. An arrangement for a concrete arch dam alternative at Watana is presented in Figure B.20. The results of this analysis indicated that the cost of the embankment dam was somewhat lower than the arch dam, even though the concrete cost rates used were significantly lower than those used for the Devil Canyon Dam. This preliminary evaluation did not indicate any overall cost savings in the project in spite of some savings in the earthworks and concrete structures for the concrete dam layout. A review of the overall construction schedule indicated a minimal savings in time for the concrete dam project.

Based on the above and the likelihood that the cost of the arch dam would increase relative to that of the embankment dam, the arch dam alternative was eliminated from further consideration.

(ii) <u>Concrete-face Rockfill Type Dam</u> (to be written)

(iii) <u>Selection of Dam Type</u>

Selection of the configuration of the embankment dam cross-section was undertaken within the context of the following basic considerations:

- The availability of suitable construction materials within economic haul distance, particularly core material;

- The requirement that the dam be capable of withstanding the effects of a significant earthquake shock (Reference 2) as well as the static loads imposed by the reservoir and its own weight;

- The relatively limited construction season available for placement of compacted fill materials.

The main dam would consist of a compacted core protected by fine and coarse filter zones on both the upstream and downstream slopes of the core. The upstream and downstream outer supporting fill zones would contain relatively free draining compacted gravel or rockfill, providing stability to the overall embankment structure. The location and inclination of the core is fundamental to the design of the embankment. Two basic alternatives exist in this regard:

- A vertical core located centrally within the dam; and
- An inclined core with both faces sloping upstream.

A central vertical core was chosen for the embankment based on a review of precedent design and the nature of the available impervious material.

The exploration program undertaken during 1980-81 indicated that adequate quantities of materials suitable for dam construction were located within reasonable haul distance from the site. The well-graded silty sand material is considered the most promising source of impervious fill. Compaction tests indicate a natural moisture content slightly on the wet side of optimum moisture content, so that control of moisture content will be critical in achieving a dense core with high shear strength.

Potential sources for the upstream and downstream shells include either river gravel from borrow areas along the Susitna River or compacted rockfill from quarries or excavations for spillways.

During the intermediate review process, the upstream slope of the dam was flattened from 2.5H:1V used during the initial review to 2.75H:1V. This slope was based on a conservative estimate of the effective shear strength parameters of the available construction materials, as well as a conservative allowance in the design for the effects of earthquake loadings on the dam.

During the final review stage, the exterior upstream slope of the dam was steepened from 2.75H:1V to 2.4H:1V, reflecting the results of the preliminary static and dynamic design analyses being undertaken at the same time as the general arrangement studies. As part of the final review, the volume of the dam with an upstream slope of 2.4H:1V was computed for four alternative dam axes. The location of these alternative axes are shown on Figure B.21. The dam volume associated with each of the four alternative axes is listed below:

Alternative Axis Number	Total Volume (million yd^3)
1	69.2
2	71.7
3	69.3
4	71.9

A section with a 2.4H:1V upstream slope and a 2H:1V downstream slope located on alternative axis number 3 was used for the final review of alternative schemes.

(e) Diversion Scheme Alternatives

The topography of the site generally dictates that diversion of the river during construction be accomplished using diversion tunnels with upstream and downstream cofferdams protecting the main construction area.

The configuration of the river in the vicinity of the site favors location of the diversion tunnels on the right bank, since the tunnel length for a tunnel on the left bank would be approximately 2,000 feet greater. In addition, rock conditions on the right bank are more favorable for tunneling and excavation of intake and outlet portals.

(i) Design Flood for Diversion

The recurrence interval of the design flood for diversion is generally established based on the characteristics of the flow regime of the river, the length of the construction period for which diversion is required and the probable consequences of overtopping of the cofferdams. Design criteria and experience from other projects similar in scope and nature have been used in selecting the diversion design flood.

At Watana, damage to the partially completed dam could be significant, or more importantly would probably result in at least a one-year delay in the completion schedule. A preliminary evaluation of the construction schedule indicates that the diversion scheme would be required for 4 or 5 years until the dam is of sufficient height to permit initial filling of the reservoir. A design flood with a return frequency of 1:50 years was selected based on experience and practice with other major hydroelectric projects. This approximates a 90 percent probability that the cofferdam will not be overtopped during the 5-year construction period. The diversion design flood together with average flow characteristics of the river significant to diversion are presented below:

Average annual flow	7,940 cfs
Maximum average monthly flow	23,100 cfs (June)
Minimum average monthly flow	890 cfs (March)
Design flood inflow (1:50 years)	81,100 cfs

(ii) Cofferdams

For the purposes of establishing the overall general arrangement of the project and for subsequent diversion optimization studies, the upstream cofferdam section adopted comprises an initial closure dam structure approximately 30 feet high placed in the wet.

(iii) Diversion Tunnels

Concrete-lined tunnels and unlined rock tunnels were compared. Preliminary hydraulic studies indicated that the design flood routed through the diversion scheme would result in a design discharge of approximately 80,500 cfs. For concrete-lined tunnels, design velocities of the order of 50 ft/s have been used in several projects. For unlined tunnels, maximum design velocities ranging from 10 ft/s in good quality rock to 4 ft/s in less competent material are typical. Thus, the volume of material to be excavated using an unlined tunnel would be at least 5 times that for a lined tunnel. The reliability of an unlined tunnel is more dependent on rock conditions than is a lined tunnel, particularly given the extended period during which the diversion scheme is required to operate. Based on these considerations, given a considerably higher cost, together with the somewhat questionable feasibility of four unlined tunnels with diameters approaching 50 feet in this type of rock, the unlined tunnels have been eliminated.

The following alternative lined tunnel schemes were examined as part of this analysis:

- Pressure tunnel with a free outlet;
- Pressure tunnel with a submerged outlet; and
- Free flow tunnel.

(iv) Emergency Release Facilities

The emergency release facilities influenced the number, type, and arrangement of the diversion tunnels selected for the final scheme.

At an early stage of the study, it was established that some form of low level release facility was required to permit lowering of the reservoir in the event of an extreme emergency, and to meet instream flow requirements during filling of the reservoir. The most economical alternative available would involve converting one of the diversion tunnels to permanent use as a low level outlet facility. Since it would be necessary to maintain the diversion scheme in service during construction of the emergency facilities outlet works, two or more diversion tunnels would be required. The use of two diversion tunnels also provides an additional measure of security to the diversion scheme in case of the loss of service of one tunnel.

The low level release facilities will be operated for approximately three years during filling of the reservoir. Discharge at high heads usually requires some form of energy dissipation prior to returning the flow to the river. Given the space restrictions imposed by the size of the diversion tunnel, it was decided to utilize a double expansion system constructed within the upper tunnel.

(v) Optimization of Diversion Scheme

Given the considerations described above relative to design flows, cofferdam configuration, and alternative types of tunnels, an economic study was undertaken to determine the optimum combination of upstream cofferdam height and tunnel diameter.

Capital costs were developed for three heights of upstream cofferdam embankment with a 30-foot-wide crest and exterior slopes of 2H:1V. A freeboard allowance of 5 feet for settlement and wave runup and 10 feet for the effects of downstream ice jamming on tailwater elevations was adopted.

Capital costs for the 4,700 foot long tunnel alternatives included allowances for excavation, concrete liner, rock bolts, and steel supports. Costs were also developed for the upstream and downstream portals, including excavation and support. The cost of intake gate structures and associated gates was determined not to vary significantly with tunnel diameter and was excluded from the analysis.

Curves of headwater elevation versus tunnel diameter for the various tunnel alternatives with submerged and free outlets are presented in Figure B.22. The relationship between capital cost and crest elevation for the upstream cofferdam is shown in Figure B.23. The capital cost for various tunnel diameters with free and submerged outlets is given in Figure B.24.

The results of the optimization study are presented in Figure B.25 and indicate the following optimum solutions for each alternative.

Type of Tunnel	Diameter (feet)	Cofferdam Crest Elevation (ft)	Total Cost ($)
Two pressure tunnels	30	1595	66,000,000
Two free flow tunnels	32.5	1580	68,000,000
Two free flow tunnels	35	1555	69,000,000

The cost studies indicate that a relatively small cost differential (4 to 5 percent) separates the various alternatives for tunnel diameter from 30 to 35 feet.

(vi) Selected Diversion Scheme

An important consideration at this point is ease of cofferdam closure. For the pressure tunnel scheme, the invert of the tunnel entrance is below riverbed elevation, and once the tunnel is complete diversion can be accomplished with a closure dam section approximately 10 feet high. The free flow tunnel scheme, however, requires a tunnel invert approximately 30 feet above the riverbed level, and diversion would involve an end-dumped closure section 50 feet high. The velocities of flows which would overtop the cofferdam before the water levels were raised to reach the tunnel invert level would be prohibitively higher resulting in complete erosion of the cofferdam and hence the dual free flow tunnel scheme was dropped from consideration.

Based on the preceeding considerations, a combination of one pressure tunnel and one free flow tunnel (or pressure tunnel with free outlet) was adopted. This will permit initial diversion to be made using the lower pressure tunnel, thereby simplifying the critical closure operation and avoiding potentially serious delays in the schedule. Two alternatives were re-evaluated as follows:

Tunnel Diameter (feet)	Upstream Cofferdam	
	Crest Elevation (feet)	Approximate Height (feet)
30	1595	150
35	1555	110

More detailed layout studies indicated that the higher cofferdam associated with the 30 foot diameter tunnel alternative would require locating the inlet portal further upstream into "The Fins" shear zone. Since good rock conditions for portal construction are essential and the 35 foot diameter tunnel alternative would permit a portal location downstream of "The Fins", this latter alternative was adopted. As noted in (v), the overall cost difference was not significant in the range of tunnel diameters considered, and the scheme incorporating two 35 foot diameter tunnels with an upstream cofferdam crest elevation of 1555 was incorporated as part of the selected general arrangement.

(f) Spillway Facilities Alternatives

As discussed in Subsection (c) above, the project has been designed to safely pass floods with the following return frequencies:

Flood	Frequency	Total Spillway Discharge (cfs)
Spillway Design	1:10,000 years	145,000
Probable Maximum	--	310,000

Discharge of the spillway design flood will require a gated service spillway on either the left or right bank. Three basic alternative spillway types were examined:

- Chute spillway with flip bucket;
- Chute spillway with stilling basin; and
- Cascade spillway.

Consideration was also given to combinations of these alternatives with or without supplemental facilities such as valved tunnels and an emergency spillway fuse plug for handling the PMF discharge.

Clearly, the selected spillway alternatives will greatly influence and be influenced by the project general arrangement.

(i) Energy Dissipation

The two chute spillway alternatives considered achieved effective energy dissipation either by means of a flip bucket which directs the spillway discharge in the form of a free-fall jet into a plunge pool well downstream from the dam or a stilling basin at the end of the chute which dissipates energy in a hydraulic jump. The cascade type spillway limits the free fall height of the discharge by utilizing a series of 20 to 50 foot steps down to river level, with energy dissipation at each step.

All spillway alternatives were assumed to incorporate a concrete ogee type control section controlled by fixed roller vertical lift gates. Chute spillway sections were assumed to be concrete-lined, with ample provision for air entrainment in the chute to prevent cavitation, and with pressure relief drains and rock anchors in the foundation.

(ii) Environmental Mitigation

During development of the general arrangements for both the Watana and Devil Canyon dams, a restriction was imposed on the amount of excess dissolved nitrogen permitted in the spillway discharges. Supersaturation occurs when aerated flows are subjected to pressures greater than 30 to 40 feet of head which forces excess nitrogen into solution. This occurs when water is subjected to the high pressures that occur in deep plunge pools or at large hydraulic jumps. The excess nitrogen would not be dissipated within the downstream Devil Canyon reservoir and a buildup of nitrogen concentration could occur throughout the body of water. It would eventually be discharged downstream from Devil Canyon with harmful effects on the fish population. On the basis of an evaluation of the related impacts and discussions with interested federal and state agencies, spillway facilities were designed to limit discharges of water from either Watana or Devil Canyon that may become supersaturated with nitrogen to a recurrence period of not less than 1:50 years.

(g) Power Facilities Alternative

Selection of the optimum power plant development involved consideration of the following:

- Location, type and size of the power plant;
- Geotechnical considerations;
- Number, type, size and setting of generating units;
- Arrangement of intake and water passages; and
- Environmental constraints.

(i) Comparison of Surface and Underground Powerhouse

Studies were carried out to compare the construction costs of a surface powerhouse and of an underground powerhouse at Watana. These studies were undertaken on the basis of preliminary conceptual layouts assuming six units and a total installed capacity of 840 MW. The comparative cost estimates for powerhouse civil works and electrical and mechanical equipment (excluding common items) indicated an advantage in favor of the underground powerhouse of \$16,300,000. The additional cost for the surface powerhouse arrangement is primarily associated with the longer penstocks and the steel linings required. Although construction cost estimates for a 1020 MW plant would be somewhat higher, the overall conclusion favoring the underground location would not change.

The underground powerhouse arrangement is also better suited to the severe winter conditions in Alaska, is less affected by river flood flows in summer, and is aesthetically less obtrusive. This arrangement has therefore been adopted for further development.

(ii) Comparison of Alternative Locations

Preliminary studies were undertaken during the development of conceptual project layouts at Watana to investigate both right and left bank locations for power facilities. The configuration of the site is such that left bank locations required longer penstock and/or tailrace tunnels and were therefore more expensive.

The location on the left bank was further rejected because of indications that the underground facilities would be located in relatively poor quality rock. The underground powerhouse was therefore located on the right bank such that the major openings lay between the two major shear features ("The Fins" and the "Fingerbuster").

(iii) Underground Openings

Because no construction adits or extensive drilling in the powerhouse and tunnel locations have been completed, it has been assumed that full concrete-lining of the penstocks and tailrace tunnels would be required. This assumption is conservative and is for preliminary design only; in practice, a large proportion of the tailrace tunnels would probably be unlined, depending on the actual rock quality encountered.

The minimum center-to-center spacing of rock tunnels and caverns has been assumed for layout studies to be 2.5 times the width or diameter of the larger excavation.

(iv) Selection of Turbines

The selection of turbine type is governed by the available head and flow. For the design head and specific speed, Francis type turbines have been selected. Francis turbines have a reasonably flat load-efficiency curve over a range from about 50 percent to 115 percent of rated output with peak efficiency of about 92 percent.

The number and rating of individual units is discussed in detail in Subsection (b) above. The final selected arrangement comprised six units producing 170 MW each, rated at minimum reservoir level (from reservoir simulation studies) in the peak demand month (December) at full gate. The unit output at best efficiency and a rated head of 680 feet is 181 MW.

(v) Transformers

The selection of transformer type, size, location and step-up rating is summarized below:

- Single phase transformers are required because of transport limitations on Alaskan roads and railways;

- Direct transformation from 15 kV to 345 kV is preferred for overall system transient stability;

- An underground transformer gallery has been selected for minimum total cost of transformers, cables, bus, and transformer losses; and

- A grouped arrangement of three sets of three single-phase transformers for each set of two units has been selected (a total of nine transformers) to reduce the physical size of the transformer gallery and to provide a transformer spacing comparable with the unit spacing.

(vi) Power Intake and Water Passages

The power intake and approach channel are significant items in the cost of the overall power facilities arrangement. The size of the intake is controlled by the number and minimum spacing between the penstocks, which in turn is dictated by geotechnical considerations.

The preferred penstock arrangement comprises six individual penstocks, one for each turbine. With this arrangement, no inlet valve is required in the powerhouse since turbine dewatering can be performed by closing the control gate at the intake and draining the penstocks and scroll case through a valved bypass to the tailrace. An alternative a.rangement with three penstocks was considered in detail to \ssess any possible advantages. This scheme would requi.e a bifurcation and two inlet valves on each penstock and extra space in the powerhouse to accommodate the inlet valves. Estimates of relative cost differences are summarized below:

Item	Cost Difference ($ x 10^6) 6 Penstocks	3 Penstocks
Intake	Base Case	-20.0
Penstocks	0	- 3.0
Bifurcations	0	+ 3.0
Valves	0	+ 4.0
Powerhouse	0	+ 8.0
Capitalized Value of Extra Head Loss	0	+ 6.0
Total	0	- 2.0

Despite a marginal saving of $2 million (or less than 2 percent in a total estimated cost of $120 million) in favor of three penstocks, the arrangement of six individual penstocks has been retained. This arrangement provides improved flexibility and security of operation.

The preliminary design of the power facilities involves two tailrace tunnels leading from a common surge chamber. An alternative arrangement with a single tailrace tunnel was also considered, but no significant cost saving was apparent.

Optimization studies on all water passages were carried out to determine the minimum total cost of initial construction plus the capitalized value of anticipated energy losses caused by conduit friction, bends and changes of section. For the penstock optimization, the construction costs of the intake and approach channel were included as a function of the penstock diameter and spacing. Similarly, in the optimization studies for the tailrace tunnels the costs of the surge chamber were included as a function of tailrace tunnel diameter.

(vii) Environmental Constraints

Apart from the potential nitrogen supersaturation problem discussed, the major environmental constraints on the design of the power facilities are:

- Control of downstream river temperatures; and
- Control of downstream flows.

The intake design has been modified to enable power plant flows to be drawn from the reservoir at four different levels throughout the anticipated range of reservoir drawdown for energy production in order to control the downstream river temperatures within acceptable limits.

Minimum flows at Gold Creek during the critical summer months have been studied to mitigate the project impacts on salmon spawning downstream of Devil Canyon. These minimum flows represent a constraint on the reservoir operation and influence the computation of average and firm energy produced by the Susitna development.

The Watana development will be operated as a daily peaking plant for load following. The actual extent of daily peaking will be dictated by unit availability, unit size, system demand, system stability, generating costs, etc.

2.3 - Selection of Watana General Arrangement

Preliminary alternative arrangements of the Watana Project were developed and subjected to a series of review and screening processes. The layouts selected from each screening process were developed in greater detail prior to the next review and, where necessary, additional layouts were prepared combining the features of two or more of the alternatives. Assumptions and criteria were evaluated at each stage and additional data incorporated as necessary. The selection process followed the general selection methodology established for the Susitna project and is outlined below.

(a) Selection Methodology

The determination of the project general arrangement at Watana was undertaken in three distinct review stages: preliminary, intermediate, and final.

(i) Preliminary Review (completed early in 1981)

This comprised four steps:

- Step 1: Assemble available data;
  Determine design criteria; and
  Establish evaluation criteria.

- Step 2: Develop preliminary layouts and design criteria based on the above data including all plausible alternatives for the constituent facilities and structures.

- Step 3: Review all layouts on the basis of technical feasibility, readily apparent cost differences, safety, and environmental impact.

- Step 4: Select those layouts that can be identified as most favorable, based on the evaluation criteria established in Step 1, and taking into account the preliminary nature of the work at this stage.

(ii) Intermediate Review (completed by mid-1981)

This involved a series of 5 steps:

- Step 1: Review all data, incorporating additional data from other work tasks.

  Review and expand design criteria to a greater level of detail.

  Review evaluation criteria and modify, if necessary.

- Step 2: Revise selected layouts on basis of the revised criteria and additional data. Prepare plans and principal sections of layouts.

- Step 3: Prepare quantity estimates for major structures based on drawings prepared under Step 2.

  Develop a preliminary construction schedule to evaluate whether or not the selected layout will allow completion of the project within the required time frame.

  Prepare a preliminary contractor's type estimate to determine the overall cost of each scheme.

- Step 4: Review all layouts on the basis of technical feasibility, cost impact of possible unknown conditions and uncertainty of assumptions, safety, and environmental impact.

- Step 5: Select the two most favorable layouts based on the evaluation criteria determined under Step 1.

(iii) Final Review (completed early in 1982)

- Step 1: Assemble and review any additional data from other work tasks.

  Revise design criteria in accordance with additional available data.

  Finalize overall evaluation criteria.

- Step 2: Revise or further develop the two layouts on the basis of input from Step 1 and determine overall dimensions of structures, water passages, gates, and other key items.

- Step 3: Prepare quantity take-offs for all major structures.

  Review cost components within a preliminary contractor's type estimate using the most recent data and criteria, and develop a construction schedule.

  Determine overall direct cost of schemes.

- Step 4: Review all layouts on the basis of practicability, technical feasibility, cost, impact of possible unknown conditions, safety, and environmental impact.

- Step 5: Select the final layout on the basis of the evaluation criteria developed under Step 1.

(b) Design Data and Criteria

As discussed above, the review process included assembling relevant design data, establishing preliminary design criteria, and expanding and refining these data during the intermediate and final reviews of the project arrangement. The design data and design criteria which evolved through the final review are presented in Table B.26.

(c) Evaluation Criteria

The various layouts were evaluated at each stage of the review process on the basis of the criteria summarized in Table B.27. The criteria listed in Table B.27 illustrate the progressively more detailed evaluation process leading to the final selected arrangement.

(d) Preliminary Review

The development selection studies described in Section 8, Volume 1 of Reference 4, involved comparisons of hydroelectric schemes at a number of sites on the Susitna River. As part of these comparisons a preliminary conceptual design was developed for Watana incorporating a double stilling basin type spillway.

Eight further layouts were subsequently prepared and examined for the Watana project during this preliminary review process in addition to the scheme shown on Figure B.7. These eight layouts are shown in schematic form on Figure B.25. Alternative 1 of these layouts was the scheme recommended for further study in the Development Section Report, Reference 1.

This section describes the preliminary review undertaken of alternative Watana layouts.

(i) Basis of Comparison of Alternatives

Although it was recognized that provision would have to be made for downstream releases of water during filling of the reservoir and for emergency reservoir drawdown, these features were not incorporated in these preliminary layouts. These facilities would either be interconnected with the diversion tunnels or be provided for separately. Since the system selected would be similar for all layouts with minimal cost differences and little impact on other structures, it was decided to exclude these facilities from overall assessment at this early stage.

Ongoing geotechnical explorations had identified the two major shear zones crossing the Susitna River and running roughly parallel in the northwest direction. These zones enclose a stretch of watercourse approximately 4500 feet in length. Preliminary evaluation of the existing geological data indicated highly fractured and altered materials within the actual shear zones which would pose serious problems for conventional tunneling methods and would be unsuitable for founding of massive concrete structures. The originally proposed dam axis was located between these shear zones, and since no apparent major advantage appeared to be gained from large changes in the dam location, layouts generally were kept within the confines of these bounding zones.

An earth and rockfill dam was used as the basis for all layouts. The downstream slope of the dam was assumed as 2H:1V in all alternatives and upstream slopes varying between 2.5H:1V and 2.25H:1V were examined in order to determine the influence of variance in the dam slope on the congestion of the layout. In all preliminary arrangements except the one shown on Plate 8.2, cofferdams were incorporated within the body of the main dam.

Floods greater than the routed 1:10,000 year spillway design flood and up to the probable maximum flood were assumed to be passed by surcharging the spillways, except in cases where an unlined cascade or stilling basin type spillway served as the sole discharge facility. In such instances, under large surcharges, these spillways would not act as efficient energy dissipators but would be drowned out, acting as steep open channels with the possibility of their total destruction. In order to avoid such an occurrence the design flood for these latter spillways was considered as the routed probable maximum flood.

On the basis of information existing at the time of the preliminary review, it appeared that an underground powerhouse could be located on either side of the river. A surface powerhouse on the right bank appeared feasible but was precluded from the left bank by the close proximity of the downstream toe of the dam and the adjacent broad shear zone. Locating the powerhouse further downstream would require tunneling across the shear zone, which would be expensive, and would require excavating a talus slope. Furthermore, it was found that a left bank surface powerhouse would either interfere with a left bank spillway or would be directly impacted by discharges from a right bank spillway.

(ii) Description of Alternative

- Double Stilling Basin Scheme

The scheme as shown on Figure B.7 has a dam axis location similar to that originally proposed by the COE, and a right bank double stilling basin spillway. The spillway follows the shortest line to the river, avoiding interference with the dam and discharging downstream almost parallel to the flow into the center of the river. A substantial amount of excavation is required for the chute and stilling basins, although most of this material could probably be used in the dam. A large volume of concrete is also required for this type of spillway, resulting in a spillway system that would be very costly. The maximum head dissipated within each stilling basin is approximately 450 feet. Within world experience, cavitation and erosion of the chute and basins should not be a problem if the structures are properly designed. Extensive erosion downstream would not be expected.

The diversion follows the shortest route, cutting the bend of the river on the right bank, and has inlet portals as far upstream as possible without having to tunnel through "The Fins". It is possible that the underground powerhouse is in the area of "The Fingerbuster", but the powerhouse could be located upstream almost as far as the system of drain holes and galleries just downstream of the main dam grout curtain.

- Alternative 1

This alternative is that recommended for further study in Reference 5 and is similar to the layout described above except that the right side of the dam has been rotated clockwise, the axis relocated upstream, and the spillway changed to a chute and flip bucket. The revised dam alignment resulted in a slight reduction in total dam volume compared to the above alternative. A localized downstream curve was introduced in the dam close to the right abutment in order to reduce the length of the spillway. The alignment of the spillway is almost parallel to the downstream section of the river and it discharges into a pre-excavated plunge pool in the river approximately 800 feet downstream from the flip bucket. This type of spillway should be considerably less costly than one incorporating a stilling basin, provided that excessive pre-excavation of bedrock within the plunge pool area is not required. Careful design of the bucket will be required, however, to prevent excessive erosion downstream causing undermining of the valley sides and/or build up of material downstream which could cause elevation of the tailwater levels.

- Alternatives 2 through 2D

Alternative 2 consists of a left bank cascade spillway with the main dam axis curving downstream at the abutments. The cascade spillway would require an extremely large volume of rock excavation but it is probable that most of this material, with careful scheduling, could be used in the dam. The excavation would cross "The Fingerbuster" and extensive dental concrete would be required in that area. In the upstream portion of the spillway, velocities would be relatively high because of the narrow configuration of the channel, and erosion could take place in this area in proximity to the dam. The discharge from the spillway enters the river perpendicular to the general flow but velocities would be relatively low and should not cause substantial erosion problems. The powerhouse is in the most suitable location for a surface alternative where the bedrock is close to the surface and the overall rock slope is approximately 2H:1V.

Alternative 2A is similar to Alternative 2 except that the upper end of the channel is divided and separate control structures are provided. This division would allow the use of one structure or upstream channel while maintenance or remedial work is being performed on the other.

Alternative 2B is similar to Alternative 2 except that the cascade spillway is replaced by a double stilling basin type structure. This spillway is somewhat longer than the similar type of structure on the right bank in the alternative described above. However, the slope of the ground is less than the rather steep right bank and may be easier to construct, a factor which may partly mitigate the cost of the longer structure. The discharge is at a sharp angle to the river and more concentrated than the cascade, which could cause erosion of the opposite bank.

Alternative 2C is a derivative of 2B with a similar arrangement, except that the double stilling basin spillway is reduced in size and augmented by an additional emergency spillway in the form of an inclined, unlined rock channel. Under this arrangement the concrete spillway acts as the main spillway, passing the 1:10,000 year design flood with greater flows passed down the unlined channel which is closed at its upstream end by an erodable fuse plug. The problems of erosion of the opposite bank still remain, although these could be overcome by excavation and/or slope protection. Erosion of the chute would be extreme for significant flows, although it is highly unlikely that this emergency spillway would ever be used.

Alternative 2D replaces the cascade of Alternative 2 with a lined chute and flip bucket. The comments relative to the flip bucket are the same as for Alternative 1 except that the left bank location in this instance requires a longer chute, partly offset by lower construction costs because of the flatter slope. The flip bucket discharges into the river at an angle which may cause erosion of the opposite bank. The underground powerhouse is located on the right bank, an arrangement which provides an overall reduction of the length of the water passages.

- <u>Alternative 3</u>

This arrangement has a dam axis location slightly upstream from Alternative 2, but retains the downstream curve at the abutments. The main spillway is an unlined rock cascade on the left bank which passes the design flood. Discharges beyond the 1:10,000 year flood would be discharged through the auxiliary concrete-lined chute and flip bucket spillway on the right bank. A gated control structure is provided for this auxiliary spillway which gives it the flexibility to be used as a back-up if maintenance should be required on the main spillway. Erosion of the cascade may be a problem, as mentioned previously, but erosion downstream should be a less important consideration because of the low unit discharge and the infrequent operation of the spillway. The diversion tunnels are situated in the right abutment, as with previous arrangements, and are of similar cost for all these alternatives.

- Alternative 4

This alternative involves rotating the axis of the main dam so that the left abutment is relocated approximately 1000 feet downstream from its Alternative 2 location. The relocation results in a reduction in the overall dam quantities but would require siting the impervious core of the dam directly over the "Fingerbuster" shear zone at maximum dam height. The left bank spillway, consisting of chute and flip bucket, is reduced in length compared to other left bank locations, as are the power facility water passages. The diversion tunnels are situated on the left bank; there is no advantage to a right bank location, since the tunnels are of similar length owing to the overall downstream relocation of the dam. Spillways and power facilities would also be lengthened by a right bank location with this dam configuration.

- Selection of Schemes for Further Study

A basic consideration during design development was that the main dam core should not cross the major shear zones because of the obvious problems with treatment of the foundation. Accordingly, there is very little scope for realigning the main dam apart from a slight rotation to place it more at right angles to the river.

Location of the spillway on the right bank results in a shorter distance to the river and allows discharges almost parallel to the general direction of river flow. The double stilling basin arrangement would be extremely expensive, particularly if it must be designed to pass the probable maximum flood. An alternative such as 2C would reduce the magnitude of design flood to be passed by the spillway but would only be acceptable if an emergency spillway with a high degree of operational predictability could be constructed. A flip bucket spillway on the right bank, discharging directly down the river, would appear to be an economic arrangement, although some scour might occur in the plunge pool area. A cascade spillway on the left bank could be an acceptable solution providing most of the excavated material could be used in the dam, and adequate rock conditions exist.

The length of diversion tunnels can be decreased if they are located on the right bank. In addition, the tunnels would be accessible by a preliminary access road from the north, which is the most likely route. This location would also avoid the area of "The Fingerbuster" and the steep cliffs which would be encountered on the left side close to the downstream dam toe.

The underground configuration assumed for the powerhouse in these preliminary studies allows for location on either side of the river with a minimum of interference with the surface structures.

Four of the preceding layouts, or variations of them, were selected for further study:

- A variation of the double stilling basin scheme, but with a single stilling basin main spillway on the right bank, a rock channel and fuse plug emergency spillway, a left bank underground powerhouse and a right bank diversion scheme;

- Alternative 1 with a right bank flip bucket spillway, an underground powerhouse on the left bank, and right bank diversion;

- A variation of Alternative 2 with a reduced capacity main spillway and a right bank rock channel with a fuse plug serving as an emergency spillway; and

- Alternative 4 with a left bank rock cascade spillway, a right bank underground powerhouse, and a right bank diversion.

(e) Intermediate Review

For the intermediate review process, the four schemes selected as a result of the preliminary review were examined in more detail and modified. A description of each of the schemes is given below and shown on Figures B.27 through B.32. The general locations of the upstream and downstream shear zones shown on these plates are approximate and have been refined on the basis of subsequent field investigations for the proposed project.

(i) Description of Alternative Schemes

The four schemes are shown on Figures B.27 through B.32.

- Scheme WP1 (Figure B.27)

  This scheme is a refinement of Alternative 1. The upstream slope of the dam is flattened from 2.5:1 to 2.75:1. This conservative approach was adopted to provide an assessment of the possible impacts on project layout of conceivable measures which prove necessary in dealing with severe earthquake design conditions. Uncertainty with regard to the nature of river alluvium also led to the location of the cofferdams outside the limits of the main dam embankment. As a result of these conditions, the intake portals of the diversion tunnels on the right bank are also moved upstream from "The Fins". A chute spillway with a flip bucket is located on the right bank. The underground powerhouse is located on the left bank.

- Scheme WP2 (Figures B.29 and B.30)

  This scheme is derived from the double stilling basin layout. The main dam and diversion facilities are similar to Scheme WP1 except that the downstream cofferdam is relocated further downstream from the spillway outlet and the diversion tunnels are correspondingly extended. The main spillway is located on the right bank, but the two stilling basins of the preliminary DSR scheme are combined into a single stilling basin at the river level. An emergency spillway is also located on the right bank and consists of a channel excavated in rock, discharging downstream from the area of the relict channel. The channel is closed at its upstream end by a compacted earthfill fuse plug and is capable of discharging the flow differential between the probable maximum flood and the 1:10,000-year design flood of the main spillway. The underground powerhouse is located on the left bank.

- Scheme WP3 (Figures B.28 and B.29)

  This scheme is similar to Scheme WP1 in all respects except that an emergency spillway is added consisting of right bank rock channel and fuse plug.

- Scheme WP4 (Figures B.31 and B.32)

  The dam location and geometry for Scheme WP4 are similar to that for the other schemes. The diversion is on the right bank and discharges downstream from the powerhouse tailrace outlet. A rock cascade spillway is located on the left bank and is served by two separate control structures with downstream stilling basins. The underground powerhouse is located on the right bank.

(ii) Comparison of Schemes

The main dam is in the same location and has the same configuration for each of the four layouts considered. The cofferdams have been located outside the limits of the main dam in order to allow more extensive excavation of the alluvial material and to ensure a sound rock foundation beneath the complete area of the dam. The overall design of the dam is conservative, and it was recognized during the evaluation that savings in both fill and excavation costs can probably be made after more detailed study.

The diversion tunnels are located on the right bank. The upstream flattening of the dam slope necessitates the location of the diversion inlets upstream from "The Fins" shear zone which would require extensive excavation and support where the tunnels pass through this extremely poor rock zone and could cause delays in the construction schedule.

A low-lying area exists on the right bank in the area of the relict channel and requires approximately a 50-foot high saddle dam for closure, given the reservoir operating level assumed for the comparison study. However, the finally selected reservoir operating level will require only a nominal freeboard structure at this location.

A summary of capital cost estimates for the four alternative schemes is given in Table B.28.

The results of this intermediate analysis indicate that the chute spillway with flip bucket (Scheme WP1) is the least costly spillway alternative.

The scheme has the additional advantage of relatively simple operating characteristics. The control structure has provision for surcharging to pass the design flood. The probable maximum flood can be passed by additional surcharging up to the crest level of the dam. In Scheme WP3 a similar spillway is provided, except that the control structure is reduced in size and discharges above the routed design flood are passed through the rock channel emergency spillway. The arrangement in Scheme WP1 does not provide a backup facility to the main spillway, so that if repairs caused by excessive plunge pool erosion or damage to the structure itself require removal of the spillway from service for any length of time, no alternative discharge facility would be available. The additional spillway of Scheme WP3 would permit emergency discharge if it were required under extreme circumstances.

The stilling basin spillway (Scheme WP2) would reduce the potential for extensive erosion downstream, but high velocities in the lower part of the chute could cause cavitation even with the provision for aeration of the discharge. This type of spillway would be very costly, as can be seen from Table B.28.

The feasibility of the rock cascade spillway is entirely dependent on the quality of the rock, which dictates the amount of treatment required for the rock surface and also the proportion of the excavated material which can be used in the dam. For determining the capital cost of Scheme WP4, conservative assumptions were made regarding surface treatment and the portion of material that would have to be wasted.

The diversion tunnels are located on the right bank for all alternatives examined in the intermediate review. For Scheme WP2, the downstream portals must be located downstream from the stilling basin, resulting in an increase of approximately 800 feet in the length of the tunnels. The left bank location of the powerhouse requires its placement close to a suspected shear zone, with the tailrace tunnels passing through this shear zone to reach the river. A longer access tunnel is also required, together with an additional 1,000 feet in the length of the tailrace. The left-side location is remote from the main access road, which will probably be on the north side of the river, as will the transmission corridor.

(iii) Selection of Schemes for Further Study

Examination of the technical and economic aspects of Scheme WP1 through WP4 indicates there is little scope for adjustment of the dam axis owing to the confinement imposed by the upstream and downstream shear zones. In addition, passage of the diversion tunnels through the upstream shear zone could result in significant delays in construction and additional cost.

From a comparison of costs in Table B.28, it can be seen that the flip bucket type spillway is the most economical, but because of the potential for erosion under extensive operation it is undesirable to use it as the only discharge facility. A mid-level release will be required for emergency drawdown of the reservoir, and use of this release as the first-stage service spillway with the flip bucket as a backup facility would combine flexibility and safety of operation with reasonable cost. The emergency rock channel spillway would be retained for discharge of flows above the routed 1:10,000-year flood.

The stilling basin spillway is very costly and the operating head of 800 feet is beyond precedent experience. Erosion downstream should not be a problem but cavitation of the chute could occur. Scheme WP2 was therefore eliminated from further consideration.

The cascade spillway was also not favored for technical and economic reasons. However, this arrangement does have an advantage in that it provides a means of preventing nitrogen supersaturation in the downstream discharges from the project which could be harmful to the fish population. A cascade configuration would reduce the dissolved nitrogen content; hence, this alternative was retained for further evaluation. The capacity of the cascade was reduced and the emergency rock channel spillway was included to take the extreme floods.

The results of the intermediate review indicated that the following components should be incorporated into any scheme carried forward for final review:

- Two diversion tunnels located on the right bank of the river;

- An underground powerhouse also located on the right bank;

- An emergency spillway, comprising a rock channel excavated on the right bank and discharging well downstream from the right abutment. The channel is sealed by an erodible fuse plug of impervious material designed to fail if overtopped by the reservoir; and

- A compacted earthfill and rockfill dam situated between the two major shear zones which traverse the project site.

As discussed above, two specific alternative methods exist with respect to routing of the spillway design flood and minimizing the adverse effects of nitrogen supersaturation on the downstream fish population. These alternatives are:

- A chute spillway with flip bucket on the right bank to pass the spillway design flood, with a mid-level release system designed to operate for floods with a frequency of up to about 1:50 years; or

- A cascade spillway on the left bank.

Accordingly, two schemes were developed for further evaluation as part of the final review process. These schemes are described separately in the paragraphs below.

(f) Final Review

The two schemes considered in the final review process were essentially derivations of Schemes WP3 and WP4.

(i) Scheme WP3A (Figure B.33)

This scheme is a modified version of Scheme WP3 described above. Because of scheduling and cost considerations, it is extremely important to maintain the diversion tunnels downstream from "The Fins." It is also important to keep the dam axis as far upstream as possible to avoid congestion of the downstream structures. For these reasons, the inlet portals to the diversion tunnels were located in the sound bedrock forming the downstream boundary of "The Fins." The upstream cofferdam and main dam are maintained in the upstream locations as shown on Figure B.33. As mentioned previously, additional criteria have necessitated modifications in the spillway configuration, and low-level and emergency drawdown outlets have been introduced.

The main modifications to the scheme are as follows:

- Main Dam

Continuing preliminary design studies and review of world practice suggest that an upstream slope of 2.4H:1V would be acceptable for the rock shell. Adoption of this slope results not only in a reduction in dam fill volume but also in a reduction in the base width of the dam which permits the main project components to be located between the major shear zones.

The downstream slope of the dam is retained as 2H:1V. The cofferdams remain outside the limits of the dam in order to allow complete excavation of the riverbed alluvium.

- Diversion

In the intermediate review arrangements, diversion tunnels passed through the broad structure of "The Fins," an intensely sheared area of breccia, gouge, and infills. Tunneling of this material would be difficult, and might even require excavation in open cut from the surface. High cost would be involved, but more important would be the time taken for construction in this area and the possibility of unexpected delays. For this reason, the inlet portals have been relocated downstream from this zone with the tunnels located closer to the river and crossing the main system of jointing at approximately 45°. This arrangement allows for shorter tunnels with a more favorable orientation of the inlet and outlet portals with respect to the river flow directions.

A separate low-level inlet and concrete-lined tunnel is provided, leading from the reservoir at approximate Elevation 1550 to downstream of the diversion plug where it merges with the diversion tunnel closest to the river. This low-level tunnel is designed to pass flows up to 6000 cfs during reservoir filling. It would also pass up to 30,000 cfs under 500-foot head to allow emergency draining of the reservoir.

Initial closure is made by lowering the gates to the tunnel located closest to the river and constructing a concrete closure plug in the tunnel at the location of the grout curtain underlying the core of the main dam. On completion of the plug, the low-level release is opened and controlled discharges are passed downstream. The closure gates within the second diversion tunnel portal are then closed and a concrete closure plug constructed in line with the grout curtain. After closure of the gates, filling of the reservoir would commence.

- Outlet Facilities

As a provision for drawing down the reservoir in case of emergency, a mid-level release is provided. The intake to these facilities is located at depth adjacent to the power facilities intake structures. Flows would then be passed downstream through a concrete-lined tunnel, discharging beneath the downstream end of the main spillway flip bucket. In order to overcome potential nitrogen supersaturation problems, Scheme WP3A also incorporates a system of fixed cone valves at the downstream end of the outlet facilities. The valves were sized to discharge in conjunction with the powerhouse operating at 7000 cfs capacity (flows up to the equivalent routed 50-year flood). Six cone valves are required, located on branches off a steel manifold and protected by individual upstream closure gates. The valves are partly incorporated into the mass concrete block forming the flip bucket of the main spillway. The rock downstream is protected from erosion by a concrete facing slab anchored back to the sound bedrock.

- Spillways

As discussed above, the designed operation of the main spillway facilities was arranged to limit discharges of potentially nitrogen-supersaturated water from Watana to flows having an equivalent return period greater than 1:50 years.

The main chute spillway and flip bucket discharge into an excavated plunge pool in the downstream river bed. Releases are controlled by a three-gated ogee structure located adjacent to the outlet facilities and power intake structure just upstream from the dam centerline. The design discharge is approximately 114,000 cfs, corresponding to the routed 1:10,000-year flood (145,000 cfs) reduced by the 31,000 cfs flows attributable to outlet and power facilities discharges. The plunge pool is formed by excavating the alluvial river deposits to bedrock. Since the excavated plunge pool approaches the limits of the calculated maximum scour hole, it is not anticipated that, given the infrequent discharges, significant downstream erosion will occur.

An emergency spillway is provided by means of a channel excavated in rock on the right bank, discharging well downstream from the right abutment in the direction of Tsusena Creek. The channel is sealed by an erodible fuse plug of impervious material designed to fail if overtopped by the reservoir, although some preliminary excavation may be necessary. The crest level of the plug will be set at Elevation 2230, well below that of the main dam. The channel will be capable of passing the excess discharge of floods greater than the 1:10,000-year flood up to the probable maximum flood of 310,000 cfs.

- Power Facilities

The power intake is set slightly upstream from the dam axis deep within sound bedrock at the downstream end of the approach channel. The intake consists of six units with provision in each unit for drawing flows from a variety of depths covering the complete drawdown range of the reservoir. This facility also provides for drawing water from the different temperature strata within the upper part of the reservoir and thus regulating the temperature of the downstream discharges close to the natural temperatures of the river. For this preliminary conceptual arrangement, flow withdrawals from different levels are achieved by a series of upstream vertical shutters moving in a single set of guides and operated to form openings at the required level. Downstream from these shutters each unit has a pair of wheel-mounted closure gates which will isolate the individual penstocks.

The six penstocks are 18-foot-diameter, concrete-lined tunnels inclined at 55° immediately downstream from the intake to a nearly horizontal portion leading to the powerhouse. This horizontal portion is steel-lined for 150 feet upstream from the turbine units to extend the seepage path to the powerhouse and reduce the flow within the fractured rock area caused by blasting in the adjacent powerhouse cavern.

The six 170 MW turbine/generator units are housed within the major powerhouse cavern and are serviced by an overhead crane which runs the length of the powerhouse and into the service area adjacent to the units. Switchgear, maintenance room and offices are located within the main cavern, with the transformers situated downstream in a separate gallery excavated above the tailrace tunnels. Six inclined tunnels carry the connecting bus ducts from the main power hall to the transformer gallery. A vertical elevator and vent shaft run from the power cavern to the main office building and control room located at the surface. Vertical cable shafts, one for each pair of transformers, connect the transformer gallery to the switchyard directly overhead. Downstream from the transformer gallery, the underlying draft tube tunnels merge into two surge chambers (one chamber for three draft tubes) which also house the draft tube gates for isolating the units from the tailrace. The gates are operated by an overhead traveling gantry located in the upper part of each of the surge chambers. Emerging from the ends of the chambers, two concrete-lined, low-pressure tailrace tunnels carry the discharges to the river. Because of space restrictions at the river, one of these tunnels has been merged with the downstream end of the diversion tunnel. The other tunnel emerges in a separate portal with provision for the installation of bulkhead gates.

The orientation of water passages and underground caverns is such as to avoid, as far as possible, alignment of the main excavations with the major joint sets.

- Access

Access is assumed to be from the north (right) side of the river. Permanent access to structures close to the river is by a road along the right downstream river bank and then via a tunnel passing through the concrete forming the flip bucket. A tunnel from this point to the power cavern provides for vehicular access. A secondary access road across the crest of the dam passes down the left bank of the valley and across the lower part of the dam.

(ii) Scheme WP4A (Figure B.34)

This scheme is similar in most respects to Scheme WP3A previously discussed, except for the spillway arrangements.

- Main Dam

The main dam axis is similar to that of Scheme WP3A, except for a slight downstream rotation at the left abutment at the spillway control structures.

- Diversion

The diversion and low level releases are the same for the two schemes.

- Outlet Facilities

The outlet facilities used for emergency drawdown are separate from the main spillway for this scheme. The outlet facilities consists of a low-level gated inlet structure discharging up to 30,000 cfs into the river through a concrete-lined, free-flow tunnel with a ski jump flip bucket. This facility may also be operated as an auxiliary outlet to augment the main left bank spillway.

- Spillways

The main left bank spillway is capable of passing a design flow equivalent to the 1:10,000-year flood through a series of 50-foot drops into shallow pre-excavated plunge pools. The emergency spillway is designed to operate during floods of greater magnitude up to and including the PMF.

Main spillway discharges are controlled by a broad multi-gated control structure discharging into a shallow stilling basin. The feasibility of this arrangement is governed by the quality of the rock in the area, requiring both durability to withstand erosion caused by spillway flows and a high percentage of sound rockfill material that can be used from the excavation directly in the main dam.

On the basis of the site information developed concurrently with the general arrangement studies, it became apparent that the major shear zone known to exist in the left bank area extended further downstream than initial studies have indicated. The cascade spillway channel was therefore lengthened to avoid the shear area at the lower end of the cascade. The arrangement shown on Figure B.34 for Scheme WP4A does not reflect this relocation, which would increase the overall cost of the scheme.

The emergency spillway consisting of rock channel and fuse plug is similar to that of the right bank spillway scheme.

- Power Facilities

The power facilities are similar to those in Scheme WP3A.

(iii) Evaluation of Final Alternative Schemes

An evaluation of the dissimilar features for each arrangement (the main spillways and the discharge arrangements at the downstream end of the outlets) indicates a saving in capital cost of $197,000,000, excluding contingencies and indirect cost, in favor of Scheme WP3A. If this difference is adjusted for the savings associated with using an appropriate proportion of excavated material from the cascade spillway as rockfill in the main dam, this represents a net overall cost difference of approximately $110,000,000 including contingencies, engineering, and administration costs.

As discussed above, although limited information exists regarding the quality of the rock in the downstream area on the left bank, it is known that a major shear zone runs through and is adjacent to the area presently allocated to the spillway in Scheme WP4. This would require relocating the left bank cascade spillway several hundred feet farther downstream into an area where the rock quality is unknown and the topography less suited to the gentle overall slope of the cascade. The cost of the excavation would substantially increase compared to previous assumptions, irrespective of the rock quality. In addition, the resistance of the rock to erosion and the suitability for use as excavated material in the main dam would become less certain. The economic feasibility of this scheme is largely predicated on this last factor, since the ability to use the material as a source of rockfill for the main dam represents a major cost saving.

In conjunction with the main chute spillway, the problem of the occurrence of nitrogen supersaturation can be overcome by the use of a regularly operated dispersion type valve outlet facility in conjunction with the main chute spillway. Since this scheme presents a more economic solution with fewer potential problems concerning the geotechnical aspects of its design, the right bank chute arrangement (Scheme WP3A) has been adopted as the final selected scheme.

2.4 - Selection of Devil Canyon General Arrangement

This section describes the development of the general arrangement of the Devil Canyon project. The method of handling floods during construction and subsequent project operation is also outlined in this section.

The reservoir level fluctuations and inflow for Devil Canyon will essentially be controlled by operation of the upstream Watana project. This aspect is also briefly discussed in this section.

(a) Selection of Reservoir Level

The selected normal maximum operating level at Devil Canyon dam is Elevation 1455. Studies by the USBR and COE o. the Devil Canyon Project were essentially based on a similar reservoir level which corresponds to the tailwater level selected at the Watana site. Although the narrow configuration of the Devil Canyon site and the relatively low costs involved in increasing the dam height suggest that it might be economic to do so, it is clear that the upper economic limit of reservoir level at Devil Canyon is the Watana tailrace level.

Although significantly lower reservoir levels at Devil Canyon would lead to lower dam costs, the location of adequate spillway facilities in the narrow gorge would become extremely difficult and lead to offsetting increases in cost. In the extreme case, a spillway discharging over the dam would raise concerns regarding safety from scouring at the toe of the dam which have already led to rejection of such schemes.

(b) Selection of Installed Capacity

The methodology used for the preliminary selection of installed capacity at Watana and Devil Canyon is described in Section 2.2 (b).

The decision to operate Devil Canyon primarily as a base-loaded plant was governed by the following main considerations:

- Daily peaking is more effectively performed at Watana than at Devil Canyon; and

- Excessive fluctuations in discharge from the Devil Canyon dam may have an undesirable impact on mitigation measures incorporated in the final design to project the downstream fisheries.

Given this mode of operation, the required installed capacity at Devil Canyon has been determined as the maximum capacity needed to utilize the available energy from the hydrological flows of record, as modified by the reservoir operation rule curves. In years where the energy from Watana and Devil Canyon exceeds the system demand, the usable energy has been reduced at both stations in proportion to the average net head available, assuming that flows used to generate energy at Watana will also be used to generate energy at Devil Canyon.

The total capacity required at Devil Canyon in a wet year, excluding standby and spinning reserve capacity, is summarized below. The capacity shown is based on the December 1981 medium load growth forecast.

Demand Year	Capacity MW
2002	370
2005	410
2010	507

The selected total installed capacity at Devil Canyon has been established as 600 MW for design purposes. This will provide some margin for standby during forced outage and possible accelerated growth in demand.

The major factors governing the selection of the unit size at Devil Canyon are the rate of growth of system demand, the minimum station output, and the requirement of standby capacity under forced outage conditions. The above tabulation indicates that station maximum load in December will increase by about 50 percent from 2002 to 2010 (from 370 MW to 507 MW). Station minimum output in July during the same period will vary from about 150 MW to 202 MW.

The power facilities at Devil Canyon have been developed using four units at 150 MW each. This arrangement will provide for efficient station operation during low load periods as well as during peak December loads. During final design, consideration of phasing of installed capacity to match the system demand may be desirable. However, the uncertainty of load forecasts and the additional contractual costs of mobilization for equipment installation are such that for this study it has been assumed that all units will be commissioned by 2002.

The Devil Canyon reservoir will usually be full in December; hence, any forced outage could result in spilling and a loss of available energy. The units have been rated to deliver 150 MW at maximum December drawdown occuring during an extremely dry year; this means that in an average year, with higher reservoir levels the full station output can be maintained even with one unit on forced outage.

(c) Selection of Spillway Capacity

A flood frequency of 1:10,000 years was selected for the spillway design on the same basis as described for Watana. An emergency spillway with an erodible fuse plug will also be provided to safely discharge the probable maximum flood. The development plan envisages completion of the Watana project prior to construction at Devil Canyon. Accordingly, the inflow flood peaks at Devil Canyon will be less than pre-project flood peaks because of routing through the Watana reservoir. Spillway design floods are:

Flood	Inflow Peak (cfs)
1:10,000 years	165,000
Probable Maximum	346,000

The avoidance of nitrogen supersaturation in the downstream flow for Watana also will apply to Devil Canyon. Thus, the discharge of water possibly supersaturated with nitrogen from Devil Canyon will be limited to a recurrence period of not less than 1:50 years by the use of solid cone valves similar to Watana.

(d) Main Dam Alternatives

The location of the Devil Canyon damsite ~~was examined~~ during previous studies by the USBR and COE. These studies focused on the narrow entrance to the canyon and led to the recommendation of a concrete arch dam. Notwithstanding this initial appraisal, a comparative analysis was undertaken as part of this feasibility study to evaluate the relative merits of the following types of structures at the same location:

- Thick concrete arch;
- Thin concrete arch; and
- Fill embankment.

(i) Comparison of Embankment and Concrete Type Dams

The geometry was developed for both the thin concrete arch and the thick concrete arch dam and the dams were analyzed and their behavior compared under static, hydrostatic, and seismic loading conditions. The project layouts for these arch dams were compared to a layout for a rockfill dam with its associated structures.

Consideration of the central core rockfill dam layout indicated relatively small cost differences from an arch dam cost estimate, based on a cross-section significantly thicker than the finally selected design. Furthermore, no information was available to indicate that impervious core material in the necessary quantities could be found within a reasonable distance of the damsite. The rockfill dam was accordingly dropped from further consideration. [It is further noted that since this alternative dam study, seismic analysis of the rockfill dam at Watana has resulted in an upstream slope 1:2.4, thus indicating the requirement to flatten the 1:1.25 slope adopted for the rockfill dam alternative at Devil Canyon.]

Neither of the concrete arch dam layouts were intended as the final site arrangement, but were sufficiently representative of the most suitable arrangement associated with each dam type to provide an adequate basis for comparison. Each type of dam was located just downstream from where the river enters Devil Canyon and close to the canyon's narrowest point, which is the optimum location for all types of dams. A brief description of each dam type and configuration is given below.

- Rockfill Dam

For this arrangement the dam axis is some 625 feet downstream of the crown section of the concrete dams. The assumed embankment slopes are 2.25 H:1V on the upstream face and 2H:1V on the downstream face. The main dam is continuous with the left bank saddle dam, and therefore no thrust blocks are required. The crest length is 2200 feet at Elevation 1470; the crest width is 50 feet.

The dam is constructed with a central impervious core, inclined upstream, supported on the downstream side by a semi-pervious zone. These two zones are protected upstream and downstream by filter and transition materials. The shell sections are constructed of rockfill obtained from blasted bedrock. For preliminary design all dam sections are assumed to be founded on rock; external cofferdams are founded on the river alluvium, and are not incorporated into the main dam. The approximate volume of material in the main dam is 20 million cubic yards.

A single spillway is provided on the right abutment to control all flood flows. It consists of a gate control structure and a double stilling basin excavated into rock; the chute sections and stilling basins are concrete-lined, with mass concrete gravity retaining walls. The design capacity is sufficient to pass the 1-in-10,000 year flood without damage; excess capacity is provided to pass the PMF, without damage to the main dam, by surcharging the reservoir and spillway.

The powerhouse is located underground in the right abutment. The multi-level power intake is constructed in a rock cut in the right abutment on the dam centerline, with four independent penstocks to the 150 MW Francis turbines. Twin concrete-lined tailrace tunnels connect the powerhouse to the river via an intermediate draft tube manifold.

- Thick Arch Dam

The main concrete dam would be a single center arch structure, acting partly as a gravity dam, with a vertical cylindrical upstream face and a sloping downstream face inclined at 1V:0.4H. The maximum height of the dam would be 635 feet with a uniform crest width of 30 feet, a crest length of approximately 1,400 feet, and a maximum foundation width of 225 feet. The crest elevation would be 1460. The center portion of the dam would be founded on a massive mass concrete pad constructed in the excavated river bed. This central section would incorporate the main spillway with sidewalls anchored into solid bedrock and gated orifice spillways discharging down the steeply inclined downstream face of the dam into a single large stilling basin set below river level and spanning the valley.

The main dam would terminate in thrust blocks high on the abutments. The left abutment thrust block would incorporate an emergency gated control spillway structure which would discharge into a rock channel running well downstream and terminating at a level high above the river valley.

Beyond the control structure and thrust block, a low-lying saddle on the left abutment would be closed by means of a rockfill dike founded on bedrock. The powerhouse would house four 150 MW units and will be located underground within the right abutment. The intake would be constructed integrally with the dam and connected to the powerhouse by vertical steel-lined penstocks.

The main spillway would be designed to pass the 1:10,000-year routed flood with larger floods discharged downstream via the emergency spillway.

- Thin Arch Dam

The main dam would be a two-center, double-curved arch structure of similar height to the thick arch dam, but with a 20-foot uniform crest and a maximum base width of 90 feet. The crest elevation would be 1460. The center section would be founded on a concrete pad, and the extreme upper portion of the dam would terminate in concrete thrust blocks located on the abutments.

The main spillway would be located on the right abutment and would consist of a conventional gated control structure discharging down a concrete-lined chute terminating in a flip bucket. The bucket would discharge into an unlined plunge pool excavated in the riverbed alluvium and located sufficiently downstream to prevent undermining of the dam and associated structures.

The main spillway would be supplemented by orifice type spillways located high in the center portion of the dam which would discharge into a concrete-lined plunge pool immediately downstream from the dam. An emergency spillway, consisting of a fuse plug discharging into an unlined rock channel terminating well downstream, would be located beyond the saddle dam on the left abutment.

The concrete dam would terminate in a massive thrust block on each abutment which, on the left abutment, would adjoin a rockfill saddle dam.

The main and auxiliary spillways would be designed to discharge the 1:10,000-year flood. Larger floods for storms up to the probable maximum flood would be discharged through the emergency left abutment spillway.

- Comparison of Arch Dam Types

Sand and gravel for concrete aggregates are believed to be available in sufficient quantities within economic distance from the damsite. The gravel and sands are formed from the granitic and metamorphic rocks of the area; at this time it is anticipated that they will be suitable for the production of aggregates after screening and washing.

The bedrock geology of the site is discussed in Reference 3. At this time it appears that there are no geological or geotechnical concerns that would preclude either of the dam types from consideration.

Under hydrostatic and temperature loadings, stresses within the thick arch dam would be generally lower than for the thin arch alternative. However, finite element analysis has shown that the additional mass of the dam under seismic loading would produce stresses of a greater magnitude in the thick arch dam than in the thin arch dam. If the surface stresses approach the maximum allowable at a particular section, the remaining understressed area of concrete will be greater for the thick arch, and the factor of safety for the dam would be correspondingly higher. The thin arch is, however, a more efficient design and better utilizes the inherent properties of the concrete. It is designed around acceptable predetermined factors of safety and requires a much smaller volume of concrete for the actual dam structure.

(iii) Cofferdams

As at Watana, the considerable depth of riverbed alluvium at both cofferdam sites indicates that embankment-type cofferdam structures would be the only technically and economically feasible alternative at Devil Canyon. For the purposes of establishing the overall general arrangement of the project and for subsequent diversion optimization studies, the upstream cofferdam section adopted will comprise an initial closure section approximately 20 feet high constructed in the wet, with a zoned embankment constructed in the dry. The downstream cofferdam will comprise a closure dam structure approximately 30 feet high placed in the wet. Control of underseepage through the alluvium material may be required and could be achieved by means of a grouted zone. The coarse nature of the alluvium at Devil Canyon led to the selection of a grouted zone rather than a slurry wall.

(iv) Diversion Tunnels

Although studies for the Watana project indicated that concrete-lined tunnels were the most economically and technically feasible solution, this aspect was reexamined at Devil Canyon. Preliminary hydraulic studies indicated that the design flood routed through the diversion scheme would result in a design discharge of approximately 37,800 cfs. For concrete-lined tunnels, design velocities of approximately 50 ft/s would permit the use of one concrete-lined tunnel with an equivalent diameter of 30 feet. Alternatively, for unlined tunnels a maximum design velocity of 10 ft/s in good quality rock would require four unlined tunnels, each with an equivalent diameter of 35 feet, to pass the design flow. As was the case for the Watana diversion scheme, considerations of reliability and cost were considered sufficient to eliminate consideration of unlined tunnels for the diversion scheme.

For the purposes of optimization studies, only a pressure tunnel was considered, since previous studies indicated that cofferdam closure problems associated with free-flow tunnels would more than offset their other advantages.

(iii) Optimization of Diversion Scheme

Given the considerations described above relative to design flows, cofferdam configuration, and alternative types of tunnels, an economic study was undertaken to determine the optimum combination of upstream cofferdam elevation (height) and tunnel diameter.

The thick arch arrangement did not appear to have a distinct technical advantage compared to a thin arch dam and would be more expensive because of the larger volume of concrete needed. Studies, therefore, continued on refining the feasibility of the thin arch alternative.

(ii) Concrete-face Rockfill Dam Alternative (to be written)

(e) Diversion Scheme Alternatives

In this section the selection of general arrangement and the basis for sizing of the diversion scheme are presented.

(i) General Arrangements

The steep walled valley at the site essentially dictated that diversion of the river during construction be accomplished using one or two diversion tunnels, with upstream and downstream cofferdams protecting the main construction area.

The selection process for establishing the final general arrangement included examination of tunnel locations on both banks of the river. Rock conditions for tunneling did not favor one bank over the other. Access and ease of construction strongly favored the left bank or abutment, the obvious approach being via the alluvial fan. The total length of tunnel required for the left bank is approximately 300 feet greater; however, access to the right bank could not be achieved without great difficulty.

(ii) Design Flood for Diversion

The recurrence interval of the design flood for diversion was established in the same manner as for Watana dam. Accordingly, at Devil Canyon a risk of exceedence of 10 percent per annum has been adopted, equivalent to a design flood with a 1:10-year return period for each year of critical construction exposure. The critical construction time is estimated at 2.5 years. The main dam could be subjected to overtopping during construction without causing serious damage, and the existence of the Watana facility upstream will offer considerable assistance in flow regulation in case of an emergency. These considerations led to the selection of the design flood with a return frequency of 1:25 years.

The equivalent inflow, together with average flow characteristics of the river significant to diversion, are presented below:

- Average annual flow: 9,040 cfs

- Design flood inflow (1:25 years routed through Watana reservoir): 37,800 cfs

Capital costs were developed for a range of pressure tunnel diameters and corresponding upstream cofferdam embankment crest elevations with a 30-foot wide crest and exterior slopes of 2H:1V. A freeboard allowance of 5 feet was included for settlement and wave runup.

Capital costs for the tunnel alternatives included allowances for excavation, concrete liner, rock bolts, and steel supports. Costs were also developed for the upstream and downstream portals, including excavation and support. The cost of an intake gate structure and associated gates was determined not to vary significantly with tunnel diameter and was excluded from the analysis.

The centerline tunnel length in all cases was estimated to be 2,000 feet.

Rating curves for the single-pressure tunnel alternatives are presented in Figure B.35. The relationship between capital costs for the upstream cofferdam and various tunnel diameters is given in Figure B.36.

The results of the optimization study indicated that a single 30-foot-diameter pressure tunnel results in the overall least cost (Figure B.36). An upstream cofferdam 60 feet high, with a crest elevation of 945, was carried forward as part of the selected general arrangement.

(f) Spillway Alternatives

The project spillways have been designed to safely pass floods with the following return frequencies:

Flood	Frequency	Inflow Peak (cfs)	Discharge (cfs)
Spillway Design	1:10,000 years	165,000	165,000
Probable Maximum	--	346,000	365,000

A number of alternatives were considered singly and in combination for Devil Canyon spillway facilities. These included gated orifices in the main dam discharging into a plunge pool, chute or tunnel spillways with either a flip bucket or stilling basin for energy dissipation, and open channel spillways. As described for Watana, the selection of the type of spillway was influenced by the general arrangement of the major structures. The main spillway facilities will discharge the spillway design flood through a gated spillway control structure with energy dissipation by a flip bucket which directs the spillway discharge in a free fall jet into a plunge pool in the river. As noted above, restrictions with respect to limiting nitrogen supersaturation in selecting acceptable spillway discharge structures have been applied. The various spillway arrangements developed in accordance with these considerations are discussed in Section 2.5.

(g) Power Facilities Alternatives

The selection of the optimum arrangements for the power facilities involved consideration of the same factors as described for Watana.

(i) Comparison of Surface and Underground Powerhouses

A surface powerhouse at Devil Canyon would be located either at the downstream toe of the dam or along the side of the canyon wall. As determined for Watana, costs favored an underground arrangement. In addition to cost, the underground powerhouse layout has been selected based on the following:

- Insufficient space is available in the steep-sided canyon for a surface powerhouse at the base of the dam;
- The provision of an extensive intake at the crest of the arch dam would be detrimental to stress conditions in the arch dam, particularly under earthquake loading, and would require significant changes in the arch dam geometry; and
- The outlet facilities located in the arch dam are designed to discharge directly into the river valley; these would cause significant winter icing and spray problems to any surface structure below the dam.

(ii) Comparison of Alternative Locations

The underground powerhouse and related facilities have been located on the right bank for the following reasons:

- Generally superior rock quality at depth;
- The left bank area behind the main dam thrust block is unsuitable for the construction of the power intake; and
- The river turns north downstream from the dam, and hence the right bank power development is more suitable for extending the tailrace tunnel to develop extra head.

(iii) Selection of Units

The turbine type selected for the Devil Canyon development is governed by the design head and specific speed and by economic considerations. Francis turbines have been adopted for reasons similar to those discussed for Watana in Section 2.2(g).

The selection of the number and rating of individual units is discussed in detail in Section 2.4(b). The four units will be rated to deliver 150 MW each at full gate opening and minimum reservoir level in December (the peak demand month).

(iv) Transformers

Transformer selection is similar to Watana.

(v) Power Intake and Water Passages

For flexibility of operation, individual penstocks are provided to each of the four units. Detailed cost studies showed that there is no significant cost advantage in using two larger diameter penstocks with bifurcation at the powerhouse compared to four separate penstocks.

A single tailrace tunnel with a length of 6,800 feet to develop 30 feet of additional head downstream from the dam has been incorporated in the design. Detailed design may indicate that two smaller tailrace tunnels for improved reliability may be superior to one large tunnel since the extra cost involved is relatively small. The surge chamber design would be essentially the same with one or two tunnels.

The overall dimensions of the intake structure are governed by the selected diameter and number of the penstocks and the minimum penstock spacing. Detailed studies comparing construction cost to the value of energy lost or gained were carried out to determine the optimum diameter of the penstocks and the tailrace tunnel.

(vi) Environmental Constraints

In addition to potential nitrogen-saturation problems caused by spillway operation, the major impacts of the Devil Canyon power facilities development are:

- Changes in the temperature regime of the river; and
- Fluctuations in downstream river flows and levels.

Temperature modeling has indicated that a multiple level varying the intake design at Devil Canyon would not significantly affect downstream water temperatures, since these are effectively controlled by the water released from

Watana. Consequently, the intake design at Devil Canyon incorporates a single level draw-off about 75 feet below maximum reservoir operating level (El 1455).

The Devil Canyon station will normally be operated as a base-loaded plant throughout the year, to satisfy the requirement of no significant daily variation in power flow.

2.5 General Arrangement Selection - Devil Canyon

The approach to selection of a general arrangement for Devil Canyon was a similar but simplified version of that used for Watana.

(a) Selection Methodology

Preliminary alternative arrangements of the Devil Canyon project were developed and selected using two rather than three review stages. Topographic conditions at this site limited the development of reasonably feasible layouts, and four schemes were initially developed and evaluated. During the final review, the selected layout was refined based on technical, operational and environmental considerations identified during the preliminary review.

(b) Design Data and Criteria

The design data and design criteria on which the alternative layouts were based are presented in Table B.29. Subsequent to selection of the preferred Devil Canyon scheme, the information was refined and updated as part of the on-going study program.

(c) Preliminary Review

Consideration of the options available for types and locations of various structures led to the development of four primary layouts for examination at Devil Canyon in the preliminary review phase. Previous studies had led to the selection of a thin concrete arch structure for the main dam, and indicated that the most acceptable technical and economic location was at the upstream entrance to the canyon. The dam axis has been fixed in this location for all alternatives.

(i) Description of Alternative Schemes

The schemes evaluated during the preliminary review are described below. In each of the alternatives evaluated, the dam is founded on the sound bedrock underlying the riverbed. The structure is 635 feet high, has a crest width of 20 feet, and a maximum base width of 90 feet. Mass concrete thrust blocks are founded high on the abutments, the left block extending approximately 100 feet above the existing bedrock surface and supporting the upper arches of the dam. The thrust block on the right abutment makes the cross-river profile of the dam more symmetrical and contributes to a more uniform stress distribution.

- Scheme DC1 (Figure B.37)

  In this scheme, diversion facilities comprise upstream and downstream earthfill and rockfill cofferdams and two 24-foot-diameter tunnels beneath the left abutment.

  A rockfill saddle dam occupies the lower lying area beyond the left abutment running from the thrust block to the higher ground beyond. The impervious fill cut-off for the saddle dam is founded on bedrock approximately 80 feet beneath the existing ground surface. The maximum height of this dam above the foundation is approximately 200 feet.

  The routed 1:10,000-year design flood of 135,000 cfs is passed by two spillways. The main spillway is located on the right abutment. It has a design discharge of 90,000 cfs, and flows are controlled by a three-gated ogee control structure. This discharges down a concrete-lined chute and over a flip bucket which ejects the water in a diverging jet into a pre-excavated plunge pool in the riverbed. The flip bucket is set at Elevation 925, approximately 35 feet above the river level. An auxiliary spillway discharging a total of 35,000 cfs is located in the center of the dam, 100 feet below the dam crest, and is controlled by three wheel-mounted gates. The orifices are designed to direct the flow into a concrete-lined plunge pool just downstream from the dam.

  An emergency spillway is located in the sound rock south of the saddle dam. This is designed to pass discharges in excess of the 1:10,000-year flood up to a probable maximum flood of 270,000 cfs, if such an event should ever occur. The spillway is an unlined rock channel which discharges into a valley downstream from the dam leading into the Susitna River.

  The upstream end of the channel is closed by an earthfill fuse plug. The plug is designed to be eroded if overtopped by the reservoir. Since the crest is lower than either the main or saddle dams, the plug would be washed out prior to overtopping of either of these structures.

  The underground power facilities are located on the right bank of the river, within the bedrock forming the dam abutment. The rock within this abutment is of better quality with fewer shear zones and a lesser degree of jointing than the rock on the left side of the canyon, and hence more suitable for underground excavation.

The power intake is located just upstream from the bend in the valley before it turns sharply to the right into Devil Canyon. The intake structure is set deep into the rock at the downstream end of the approach channel. Separate penstocks for each unit lead to the powerhouse.

The powerhouse contains four 150 MW turbine/generator units. The turbines are Francis type units coupled to overhead umbrella type generators. The units are serviced by an overhead crane running the length of the powerhouse and into the end service bay. Offices, the control room, switchgear room, maintenance room, etc., are located beyond the service bay. The transformers are housed in a separate upstream gallery located above the lower horizontal section of the penstocks. Two vertical cable shafts connect the gallery to the surface. The draft tube gates are housed above the draft tubes in separate annexes off the main powerhall. The draft tubes converge in two bifurcations at the tailrace tunnels which discharge, under free-flow conditions, to the river. Access to the powerhouse is by means of an unlined tunnel leading from an access portal on the right side of the canyon.

The switchyard is located on the left bank of the river just downstream from the saddle dam, and the power cables from the transformers are carried to it across the top of the dam.

- Scheme DC2 (Figure B.38)

The layout is generally similar to Scheme DC1 except that the chute spillway is located on the left side of the canyon. The concrete-lined chute terminates in a flip bucket high on the left side of the canyon which drops the discharges into the river below. The design flow is 90,000 cfs, and discharges are controlled by a 3-gated, ogee-crested-control structure similar to that for Scheme DC1 which abuts the left side thrust block.

The saddle dam axis is straight, following the shortest route between the control structure at one end and the rising ground beyond the low-lying area at the other.

- Scheme DC3 (See Figure B.39)

The layout is similar to Scheme DC1 except that the right side main spillway takes the form of a single tunnel rather than an open chute. A 2-gated, ogee--control structure is located at the head of the tunnel and discharges into an inclined shaft 45 feet diameter at its upper end. The structure will discharge up to a maximum of 90,000 cfs.

The concrete-lined tunnel narrows to 35 feet diameter and discharges into a flip bucket which directs the flows in a jet into the river below as in Scheme DC1.

An auxiliary spillway is located in the center of the dam and an emergency spillway is excavated on the left abutment.

The layout of dams and power facilities are the same as for Scheme DC1.

- Scheme DC4 (See Figure B.40)

The dam, power facilities, and saddle dam for this scheme are the same as those for Scheme DC1. The major difference is the substitution of a stilling-basin type spillway on the right bank for the chute and flip bucket. A 3-gated, ogee-control structure is located at the end of the dam thrust block and controls the discharges up to a maximum of 90,000 cfs.

The concrete-lined chute is built into the face of the canyon and discharges into a 500-feet-long by 115-feet-wide by 100-feet-high concrete stilling basin formed below river level and deep within the right side of the canyon. Central orifices in the dam and the left bank rock channel and fuse plug form the auxiliary and emergency spillways, respectively, as in the other alternative schemes.

The concrete-lined chute is built into the face of the canyon and discharges into a 500-feet-long by 115-feet-wide by 100-feet-high concrete stilling basin formed below river level and deep within the right side of the canyon. Central orifices in the dam and the left bank rock channel and fuse plug form the auxiliary and emergency spillways, respectively, as in the other alternative schemes.

The downstream cofferdam is located beyond the stilling basin, and the diversion tunnel outlets are located farther downstream to enable construction of the stilling basin.

(ii) Comparison of Alternatives

The arch dam, saddle dam, power facilities, and diversion vary only in a minor degree among the four alternatives. Thus, the comparison of the schemes rests solely on a comparison of the spillway facilities.

As can be seen from a comparison of the costs in Table B.30, the flip bucket spillways are substantially less costly to construct than the stilling-basin type of Scheme DC4. The left side spillway of Scheme DC2 runs at a sharp angle to the river and ejects the discharge jet from high on the canyon face toward the opposite side of the canyon. Over a longer period of operation, scour of the heavily jointed rock could cause undermining of the canyon sides and their subsequent instability. The possibility also exists of deposition of material in the downstream riverbed with a corresponding elevation of the tailrace. Construction of a spillway on the steep left side of the river could be more difficult than on the right side because of the presence of deep fissures and large unstable blocks of rock which are present on the left side close to the top of the canyon.

The two-right side flip bucket spillway schemes, based on either an open chute or a tunnel, take advantage of a downstream bend in the river to discharge parallel to the course of the river. This will reduce the effects of erosion but could still present a problem if the estimated maximum possible scour hole would occur.

The tunnel type spillway could prove difficult to construct because of the large diameter inclined shaft and tunnel paralleling the bedding planes. The high velocities encountered in the tunnel spillway could cause problems with the possibility of spiraling flows and severe cavitation both occuring.

The stilling basin type spillway of Scheme DC4 reduces downstream erosion problems within the canyon. However, cavitation could be a problem under the high-flow velocities experienced at the base of the chute. This would be somewhat alleviated by aeration of the flows. There is, however, little precedent for stilling basin operation at heads of over 500 feet; and even where floods of much less than the design capacity have been discharged, severe damage has occurred.

(iii) Selection of Final Scheme

The chute and flip bucket spillway of Scheme DC2 could generate downstream erosion problems which could require considerable maintenance costs and cause reduced efficiency in operation of the project at a future date. Hydraulic design problems exist with Scheme DC3 which may also have severe cavitation problems. Also, there is no cost advantage in Scheme DC3 over the open chute Scheme DC1. In Scheme DC4, the operating characteristics of a high head stilling basin are little known, and there are few examples of successful operation. Scheme DC4 also costs considerably more than any other scheme (Table B.30).

All spillways operating at the required heads and discharges will eventually cause some erosion. For all schemes, the use of solid cone valve outlet facilities in the lower portion of the dam to handle floods up to 1:50-year frequency is considered a more reasonable approach to reduce erosion and eliminate nitrogen supersaturation problems than the gated high level orifice outlets in the dam. Since the cost of the flip bucket type spillway in the scheme is considerably less than that of the stilling basin in Scheme DC4, and since the latter offers no relative operational advantage, Scheme DC1 has been selected for further study as the selected scheme.

(d) Final Review

The layout selected in the previous section was further developed in accordance with updated engineering studies and criteria. The major change compared to Scheme DC1 is the elimination of the high level gated orifices and introduction of low level solid cone valves, but other modifications that were introduced are described below.

The revised layout is shown on Figure B.41. A description of the structures is as follows.

(i) Main Dam

The maximum operating level of the reservoir was raised to Elevation 1455 in accordance with updated information relative to the Watana tailwater level. This requires raising the dam crest to Elevation 1463 with the concrete parapet wall crest at Elevation 1466. The saddle dam was raised to Elevation 1472.

(ii) Spillways and Outlet Facilities

To eliminate the potential for nitrogen supersaturation problems, the outlet facilities were designed to restrict supersaturated flow to an average recurrence interval of greater than 50 years. This led to the replacement of high level gated orifice spillway by outlet facilities incorporating 7 fixed-cone valves, 3 with a diameter of 90 inches and 4 with a diameter of 102 inches, capable of passing a design flow of 38,500 cfs.

The chute spillway and flip bucket are located on the right bank, as in Scheme DC1; however, the chute length was decreased and the elevation of the flip bucket raised compared to Scheme DC1.

More recent site surveys indicated that the ground surface in the vicinity of the saddle dam was lower than originally estimated. The emergency spillway channel was relocated slightly to the south to accommodate the larger dam.

(iii) Diversion

The previous twin diversion tunnels were replaced by a single-tunnel scheme. This was determined to provide all necessary security and will cost approximately one-half as much as the two-tunnel alternative.

(iv) Power Facilities

The drawdown range of the reservoir was reduced, allowing a reduction in height of the power intake. In order to locate the intake within solid rock, it has been moved into the side of the valley, requiring a slight rotation of the water passages, powerhouse, and caverns comprising the power facilities.

3 - DESCRIPTION OF PROJECT OPERATION

Note: Adjustments may be made to this section due to operation studies currently underway in Anchorage.

3.1 - Operation Within Railbelt Power System

A staged development is planned for implementation of Susitna power generation. The following schedule for unit start-up is proposed:

Start-up Date	Dam Site	No. and Size of Units (MW) Brought On-line	Total Susitna On-line Capacity* (MW)
1993	Watana	4 x 170	680
1994	Watana	2 x 170	1020
2002	Devil Canyon	4 x 150	1620

* Installed generating capacity.

As shown above, the first four units are scheduled to be on line at Watana in early 1993, followed by the remaining two Watana units in early 1994. Startup of all four units at Devil Canyon is planned for 2002.

Of the total project installed capacity of 1620 MW, 1280 MW were utilized as the basis for generation planning. The remaining 340 MW are planned to meet the needs for spinning reserve capacity.

This section describes the operation of the Watana and Devil Canyon power plants in the Railbelt electrical system. Under current conditions in the Railbelt, a total of nine utilities share responsibility for generation and distribution of electric power, with limited interconnections. The proposed arrangements for optimization and control of the dispatch of Susitna power to Railbelt load centers is based on the expectation that a single entity will eventually be set up for this purpose. In the year 2010 the projected Railbelt system, with Susitna on line, is projected to comprise:

Coal-fired Steam:	13 MW
Natural Gas GT:	326 MW
Diesel:	6 MW
Natural Gas CC:	317 MW
Hydropower:	1680 MW
TOTAL	2482 MW

It is important to note that the Susitna project will be the single most significant power source in the system. The dispatch and distribution of power from all sources by the most economical and reliable means is therefore essential. The general principles of reliability of plant and system operation, reservoir regulation, stationary and spinning reserve requirements, and maintenance programming are discussed in this section. Estimates of dependable capacity and annual energy production for both Watana and Devil Canyon are presented. Operating and maintenance procedures are described, and the proposed performance monitoring system for the two projects is also outlined.

3.2 - Plant and System Operation Requirements

The main function of system planning and operation control is the allocation of generating plant on a short-term operational basis so that the total system demand is met by the available generation at minimum cost consistent with the security of supply. The objectives are generally the same for long-term planning or short-term operational load dispatching, but with important differences in the latter case. In the short-term case, the actual state of the system dictates system reliability requirements, overriding economic considerations in load dispatching. An important factor arising from economic and reliability considerations in system planning and operation is the provision of stationary reserve and spinning reserve capacity. Figure B.42 shows the daily variation in demand for the Railbelt system during typical winter and summer weekdays and the seasonal variation in monthly peak demands for estimated loads in a typical year (the year 2000).

3.3 - General Power Plant and System Railbelt Criteria

The following are basic reliability standards and criteria have been adopted for planning the Susitna project.

(a) Installed Generating Capacity

Sufficient generating capacity is installed in the system to insure that the probability of occurrence of load exceeding the available generating capacity shall not be greater than one day in ten years (Loss-of-load probability - LOLP - of 0.1).

(b) Transmission System Capability

The high-voltage transmission system should be operable at all load levels to meet the following unscheduled single or double contingencies without instability, cascading or interruption of load.

- The single contingency situation is the loss of any single generating unit, transmission line, transformer, or bus (in addition to normal scheduled or maintenance outages) without exceeding the applicable emergency rating of any facility; and

- The double contingency situation is the subsequent outage of any remaining equipment, line or subsystem without exceeding the short time emergency rating of any facility.

In the single contingency situation, the power system must be capable of readjustment so that all equipment will be loaded within normal ratings, and in the double contingency situation, within emergency ratings for the probable duration of the outage.

During any contingency:

- Sufficient reactive power (MVAR) capacity with adequate controls is installed to maintain acceptable transmission voltage profiles.

- The stability of the power system is maintained without loss of load or generation during and after a three-phase fault, cleared in normal time, at the most critical location.

(c) Summary

Operational reliability criteria thus fall into four main categories:

- LOLP of 0.1, or one day in ten years, is maintained for the recommended plan of operation;

- The single and double contingency requirements are maintained for any of the more probable outages in the plant or transmission system;

- System stability and voltage regulation are assured from the electrical system studies. Detailed studies for load frequency control have not been performed, but it is expected that the stipulated criteria will be met with the more than adequate spinning reserve capacity with six units at Watana and four units at Devil Canyon; and

- The loss of all Susitna transmission lines on a single right-of-way has a low level of probability. In the event of the loss of all lines, the hydro plants at Watana and Devil Canyon are best suited to restore power supply quickly after the first line is restored since they are designed for "black start" operation. In this respect, hydro plans are superior to thermal plants because of their inherent black start capability for restoration of supply to a large system.

3.4 - Economic Dispatch of Units

A Susitna Area Control Center will be located at Watana to control both the Watana and the Devil Canyon power plants as shown in Plate 34. The control center will be linked through the supervisory system to the Central Dispatch Control Center at Willow.

The supervisory control of the entire Alaska Railbelt system will be done at the Central Dispatch Center at Willow. A high level of control automation with the aid of digital computers will be sought, but not a complete computerized direct digital control of the Watana and Devil Canyon power plants. Independent operator controlled local-manual and local-auto operations will still be possible at Watana and Devil Canyon power plants for testing/commissioning or during emergencies. The control system will be designed to perform the following functions at both power plants:

- Start/stop and loading of units by operator;
- Load-frequency control of units;
- Reservoir/water flow control;
- Continuous monitoring and data logging;
- Alarm annunciation; and
- Man-machine communication through visual display units (VDU) and console.

In addition, the computer system will be capable of retrieval of technical data, design criteria, equipment characteristics and operating limitations, schematic diagrams, and operating/maintenance records of the units.

The Susitna Area Control Center will be capable of completely independent control of the Central Dispatch Center in case of system emergencies. Similarly it will be possible to operate the Susitna units in an emergency situation from the Central Dispatch Center, although this should be an unlikely operation considering the size, complexity, and impact of the Susitna generating plants on the system.

The Central Dispatch Control Engineer decides which generating units should be operated at any given time. Decisions are made on the basis of known information, including an "order-of-merit" schedule, short-term demand forecasts, limits of operation of units, and unit maintenance schedules.

(a) Merit-Order Schedule

In order to decide which generating unit should run to meet the system demand in the most economic manner, the Control Engineer is provided with information of the running cost of each unit in the form of an "order-of-merit" schedule. The schedule gives the capacity and fuel costs for thermal units, and reservoir regulation limits for hydro plants.

(b) Optimum Load Dispatching

One of the most important functions of the Control Center is the accurate forecasting of the load demands in the various areas of the system.

Based on the anticipated demand, basic power transfers between areas, and an allowance for reserve, the planned generating capacity to be used is determined by taking into consideration the reservoir regulation plans of the hydro plants. The type and size of the units should also be taken into consideration for effective load dispatching.

In a hydro-dominated power system such as the Railbelt system would be if Susitna is developed, the hydro unit will take up a much greater part of base load operation than in a thermal dominated power system. The planned hydro units at Watana typically are well suited to load following and frequency regulation of the system and providing spinning reserve. Greater flexibility of operation was a significant factor in the selection of six units of 170 MW capacity at Watana, rather than fewer larger-size units.

(c) Operating Limits of Units

There are strict constraints on the minimum load and the loading rates of machines: to dispatch load to these machines requires a systemwide dispatch program taking these constraints into consideration. In general, hydro units have excellent startup and load following characteristics; thermal units have good part- loading characteristics.

Typical plant loading limitations are given below:

(i) Hydro Units

- Reservoir regulation constraints resulting in not-to-exceed maximum and minimum reservoir levels, daily or seasonally.

- Part loading of units is impossible in the zone of rough turbine operation (typically from above speed-no-load to 50 percent load) due to vibrations arising from hydraulic surges.

(ii) Steam Units

- Loading rates are slow (10 percent per minute).

- The units may not be able to meet a sudden steep rate of rise of load demand.

- The units have a minimum economic shutdown period (about 3 hours).

- The total cost of using conventional units includes banking, raising pressure and part-load operations prior to maximum economic operation.

(iii) Gas Turbines

- Cannot be used as spinning reserve because of very poor efficiency and reduced service life.

- Require 8 to 10 minutes for normal start-up from cold. Emergency start up times are of the order of 5 to 7 minutes.

(d) Optimum Maintenance Program

An important part of operational planning which can have a significant effect on operating costs is maintenance programming. The program specifies the times in the year and the sequence in which plant is released for maintenance.

3.5 - Unit Operation Reliability Criteria

During the operational load dispatching conditions of the power system, the reliability criteria often override economic considerations in scheduling of various units in the system. Also important in considering operational reliability are system response, load-frequency control, and spinning reserve capabilities.

(a) Power System Analyses

Load-frequency response studies determine the dynamic stability of the system due to the sudden forced outage of the largest unit (or generation block) in the system. The generation and load are not balanced, and if the pick-up rate of new generation is not adequate, loss of load will eventually result from under-voltage and under-frequency relay operation, or load-shedding. The aim of a well-designed high security system is to avoid load-shedding by maintaining frequency and voltage within the specified statutory limits.

(b) System Response and Load-Frequency Control

To meet the frequency requirements, it is necessary that the effective capacity of generating plant supplying the system at any given instant should be in excess of the load demand. In the absence of detailed studies, an empirical factor of 1-2/3 times the capacity of the largest unit in the system is normally taken as a design criterion to maintain system frequency within acceptable limits in the event of the instantaneous loss of the largest unit. It is recommended that a factor of 1-1/2 times the largest unit size be considered as a minimum for the Alaska Railbelt system, with 2 times the largest unit size as a fairly conservative value (i.e., 300 to 340 MW).

The quickest response in system generation will come from the hydro units. The large hydro units at Watana and Devil Canyon on spinning reserve can respond in the turbining mode within 30 seconds. This is one of the particularly important advantages of the Susitna hydro units. Gas turbines can only respond in a second stage operation within 5 to 10 minutes and would not strictly qualify as spinning reserve. If thermal units are run part-loaded (example, 75 percent), this would be another source of spinning reserve. Ideally, it would be advantageous to provide spinning reserve in the thermal generation as well, in order to spread spinning reserves evenly in the system, with a compromise to economic loading resulting from such an operation.

(c) Protective Relaying System and Devices

The primary protective relaying systems provided for the generators and transmission system of the Susitna project are designed to disconnect the faulty equipment from the system in the fastest possible time. Independent protective systems are installed to the extent necessary to provide a fast-clearing backup for the primary protective system so as to limit equipment damage, to limit the shock to the system and to speed restoration of service. The relaying systems are designed so as not to restrict the normal or necessary network transfer capabilities of the power system.

3.6 - Dispatch Control Centers

The operation of the Watana and Devil Canyon power plant in relation to the Central Dispatch Center can be considered to be the second tier of a three-tier control structure as follows:

- Central Dispatch Control Center (345 kV network) at Willow: manages the main system energy transfers, advises system configuration and checks overall security.

- Area Control Center (Generation connected to 345 kV system, for example, Watana and Devil Canyon): deals with the loading of generators connected directly to the 345 kV network, switching and safety precautions of local systems, checks security of interconnections to main system.

- District or Load Centers (138 kV and lower voltage networks): generation and distribution at lower voltage levels.

For the Anchorage and Fairbanks areas, the district center functions are incorporated in the respective area control centers.

Each generating unit at Watana and Devil Canyon is started up, loaded and operated and shut down from the Area Control Center at Watana according to the loading demands from the Central Dispatch Control Center with due consideration to:

- Watana reservoir regulation criteria;
- Devil Canyon reservoir regulation criteria;
- Turbine loading and de-loading rates;
- Part loading and maximum loading characteristics of turbines and generators;
- Hydraulic transient characteristics of waterways and turbines;
- Load-frequency control of demands of the system; and
- Voltage regulation requirements of the system.

The Watana Area Control Center is equipped with a computer-aided control system to efficiently carry out these functions. The computer-aided control system allows a minimum of highly trained and skilled operators to perform the control and supervision of Watana and Devil Canyon plants from a single control room. The data information and retrieval system will enable the performance and alarm monitoring of each unit individually as well as the plant/reservoir and project operation as a whole.

3.7 - Susitna Project Operation

Substantial seasonal as well as over-the-year regulation of the river flow is achieved with the two reservoirs. The simulation of the reservoirs and the power facilities at the two developments was carried out on a monthly basis to assess the energy potential of the schemes, river flows downstream and flood control possibilities with the reservoirs. The following paragraphs summarize the main features of reservoir operation.

An optimum reservoir operation was established by an iterative process to minimize net system operating costs while maximizing firm and usable energy production. Four alternative operating cases for the Watana reservoir (A, B, C, and D) were selected for study, to define the possible range of operation. Case A represents an optimum power and energy scenario, while Case D reflects a case of "no impact on downstream fisheries". Cases B and C are intermediate levels of power operation and downstream impact. These essentially define monthly minimum flows at Gold Creek that must be maintained while providing energy consistent with other project constraints. For feasibility report purposes, operation model "A" was adopted for project design. Studies with appropriate fisheries mitigation measures were developed based on Case A flows at Gold Creek. Table B.31 presents a summary of potential energy generation with different operating rules for Watana and Devil Canyon developments.

Average annual energy potential of Watana development is 3460 GWh, and that of Devil Canyon development is 3340 GWh. A frequency analysis of the river hydrology was made to derive the firm annual energy potential (or the dependable capacity) of the hydro development.

The Federal Energy Regulatory Commission (FERC) defines the dependable capacity of hydroelectric plants as: "the capacity which, under the most adverse flow conditions of record can be relied upon to carry system load, provide dependable reserve capacity, and meet firm power obligations taking into account seasonal variations and other characteristics of the load to be supplied" (1). Based on the Railbelt system studies and previous experience on large hydroelectric projects, it was assumed that a dry hydrological sequence with a recurrence period of the order of 1:50 years would constitute an adequate reliability for the Railbelt electrical system.

An analysis of annual energy potential of the reservoirs showed that the lowest annual energy generation has a recurrence frequency of 1 in 300 years. The second lowest annual energy of 5400 GWh has a recurrence frequency of 1 in 70 years. This latter figure has been adopted as the firm energy from the development.

Expressed another way, the firm energy as defined may fall short of its value by about 5 percent once in 300 years. This is a conservative interpretation of the FERC definition.

The monthly distribution of firm annual energy as simulated in the reservoir simulation has been used in system generation planning studies. Average monthly energy based on the recorded sequence hydrology is used in the economic analysis.

4 - DEPENDABLE CAPACITY AND ENERGY PRODUCTION

4.1 - Hydrology

(a) Historical Streamflow Records

Historical streamflow data are available for several gaging stations on the Susitna River and its main tributaries. Continuous gaging records were available for the following eight stations on the river and its tributaries: Maclaren River near Paxson, Denali, Cantwell, Gold Creek and Susitna stations on the Susitna River, Chulitna Station on the Chulitna River, Talkeetna on the Talkeetna River, and Skwentna on the Skwentna River. The longest period of record available is for the station at Gold Creek (32 years from 1949 to 1981). At other stations, record length varies from 6 to 23 years. Gaging was continued at all these stations as part of the project study program. A gaging station was established at the Watana damsite in 1980, and streamflow records are available for the study period. Partial streamflow records are available at several other stations on the river for varying periods; the station locations are shown in Figure B.43. It should be noted that gaging will continue as the project progresses in order to improve the streamflow record, as well as after project completion at selected sites required for project operation.

(b) Water Resources

Above its confluence with the Chulitna River, the Susitna contributes approximately 20 percent of the mean annual flow measured at Susitna Station near Cook Inlet. Figure B.44 shows how the mean annual flow of the Susitna increases towards the mouth of the river at Cook Inlet.

Seasonal variation of flow in the river is extreme and ranges from very low values in winter (October to April) to high summer values (May to September). For the Susitna River at Gold Creek, the average winter and summer flows are 2,100 and 20,250 cfs respectively, i.e., a 1 to 10 ratio. This large seasonal difference is mainly due to effects of glacial and snow melt in the summer.

The monthly average flows in the Susitna River at Gold Creek are given in Figure B.45. Some 40 percent of the streamflow at Gold Creek originates above the Denali and Maclaren gages. This catchment generally comprises the glaciers and associated high mountains. On the average, approximately 88 percent of the streamflow recorded at Gold Creek station occurs during the summer months. At higher elevations in the basin, the distribution of flows is concentrated even more in the summer months. For the Maclaren River near Paxson (Elevation 4520), the average winter and summer flows are 144 and 2,100 cfs respectively, i.e. a 1 to 15 ratio. The monthly percent of annual discharge and mean monthly discharges for the Susitna River and tributaries at the gaging stations above the Chulitna confluence are given in Table B.32.

(c) Streamflow Extension

Synthesized flows at the Watana and Devil Canyon damsites are presented in Tables B.33 and B.34. Flow duration curves based on these monthly estimates are presented for Watana and Devil Canyon damsites in Figures B.46 and B.47.

The inhouse FILLIN computer program developed by the Texas Water Development Board was used to fill in gaps in historical streamflow records at the eight continuous gaging stations. The 32 year record (up to 1981) at Gold Creek was used as the base record. The procedure adopted for filling in the data gaps uses a multisite regression technique which analyzes monthly time-series data. Flow sequences for the 32-year period were generated at the remaining seven stations. Using these flows at Cantwell station and observed Gold Creek flows, 32-year monthly flow sequences at the Watana and Devil Canyon damsites were generated on the basis of prorated drainage areas. Recorded streamflows at Watana and Devil Canyon were included in the historical record where available.

(d) Critical Streamflow Used for Dependable Capacity

[Note: This section is subject to revision after selection of minimum downstream flow in October.]

Average annual energy potential of Watana development is 3460 GWh, and that of Devil Canyon development is 3340 GWh. A frequency analysis of the river hydrology was made to derive the firm annual energy potential (or the dependable capacity) of the hydro development. Based on the Railbelt system studies and previous experience on large hydroelectric projects, it was assumed that a dry hydrological sequence with a recurrence period of the order of 1:50 years would constitute an adequate reliability for the Railbelt electrical system.

An analysis of annual energy potential of the reservoirs showed that the lowest annual energy generation has a recurrence frequency of 1 in 300 years (see Figure B.48). The second lowest annual energy of 5400 GWh has a recurrence frequency of 1 in 70 years. This latter figure has been adopted as the firm energy from the development.

Expressed another way, the firm energy, as defined, may fall short of its value by about 5 percent once in 300 years. This is a conservative interpretation of the FERC definition of dependable capacity.

(e) Floods

The most common causes of flood peaks in the Susitna River Basin are snowmelt or a combination of snowmelt and rainfall over a large area. Annual maximum peak discharges generally occur between May and October with the majority (approximately 60 percent) occurring in June. Some of the annual maximum flood peaks have also occurred in August or later and are the result of heavy rains over large areas augmented by significant snowmelt from higher elevations and glacial runoff. Table B.35 presents selected flood peaks recorded at different gaging stations.

A regional flood peak and volume frequency analysis was carried out using the recorded floods in the Susitna River and its principal tributaries. These analyses were conducted for two different time periods. The first period, after the ice breakup and before freezeup (May through October), contains the largest floods which must be accommodated by the project. The second period represents that portion of time during which ice conditions occur in the river (October through May). These floods, although smaller, can be accompanied by ice jamming and must be considered during the construction phase of the project in planning the design of cofferdams for river diversion.

A set of multiple linear regression equations were developed using physiographic basin parameters such as catchment area, stream length, precipitation, snowfall amounts, etc., to estimate flood peaks at ungaged sites in the basin. In conjunction with the analysis of shapes and volumes of recorded large floods at Gold Creek, a set of project design flood hydrographs of different recurrence intervals were developed (see Figures B.49 and B.50).

The results of the above analysis were used for estimating flood hydrographs at the damsites and ungaged streams and rivers along the access road alignments for design of spillways, culverts, etc. Table B.36 lists mean annual, 50-, 100-, and 10,000-year floods at the Watana and Devil Canyon damsites and at the Gold Creek gage. The proposed reservoirs at Watana and Devil Canyon would be classified as "large" and with "high hazard potential" according to the guidelines for safety inspection of dams laid out by the Corps of Engineers. This would indicate the need for the probable maximum flood (PMF) to be considered in the evaluation of the proposed projects. Estimated peak discharges during the PMF at selected locations are included in Table B.36, and the PMF hydrograph is presented in Figure B.50.

(f) Flow Adjustments

Evaporation from the proposed Watana and Devil Canyon Reservoirs has been evaluated to determine its significance. Evaporation is influenced by air and water temperatures, wind, atmospheric pressure, and dissolved solids within the water. However, the evaluation of these factors' effects on evaporation is difficult because of their interdependence on each other. Consequently, more simplified methods were preferred and have been utilized to estimate evaporation losses from the two reservoirs.

The monthly evaporation estimates for the reservoirs are presented in Table B.37. The estimates indicate that evaporation losses will be less than or equal to additions due to precipitation on the reservoir surface. Therefore, a conservative approach was taken, with evaporation losses and precipitation gains neglected in the energy calculations.

Leakage is not expected to result in significant flow losses. Seepage through the relict channel is estimated as less than one-half of one percent of the average flow and therefore has been neglected in the energy calculations to date. This approach will be reviewed when further investigations of the relict channel are completed.

Minimum flow releases are required throughout the year to maintain downstream river stages. The most significant factor in determining the minimum flow value is the maintenance of downstream fisheries. The monthly flow requirements that were used in determination of project energy potential are given in Table B.38.

The numbers shown in Table B.38 represent the minimum stream flow required at Gold Creek. These requirements would remain constant for all phases of project development. The actual flows released from the project at Watana (when Watana is operating alone) and at Devil Canyon (for combined operation of both dams) will be less than the required Gold Creek flows, prorated on the basis of streamflow contributions from the intervening basin area. Tables B.39 and B.40 give the typical minimum required flow releases at Watana and Devil Canyon for a 32-year period of record.

After completion of Devil Canyon, flow releases from Watana will be regulated by system operation requirements. Because the tailwater of the Devil Canyon reservoir will extend upstream to the Watana tailrace, there will be no release requirements for streamflow maintenance of Watana for the Watana/Devil Canyon combined operating configuration.

Existing water rights in the Susitna Basin were investigated to determine impacts on downstream flow requirements. Based on inventory information provided by the Alaska Department of Natural Resources, it was determined that existing water users will not be affected by the project. A listing of all water appropriations located within one mile of the Susitna River is provided in Table B.41.

4.2 - Reservoir Data

(a) Reservoir Storage

Gross storage volume of the Watana reservoir at its normal maximum operating level of 2185 feet is 9.47 million ac/ft, which is about 1.6 times the mean annual flow (MAF) at the damsite. Live storage in the reservoir is about 4.3 million ac/ft (75 percent of MAF). Devil Canyon reservoir has a gross storage of about 1.1 million ac/ft and live storage of 0.34 million ac/ft.

The area-capacity curves for the Watana and Devil Canyon reservoirs are provided in Figure B.51 and Figure B.52, respectively.

(b) Rule Curves

Operation of the reservoirs for energy production is based on target water surface levels set for the end of each month. The target level represents that level below which no energy beyond firm energy can be produced. In other words, if the reservoir level drops below the target only firm energy will be produced. In wetter years when the reservoir level surpasses the target level, energies greater than firm energy can be produced, but only as great as the system energy demand allows.

With a reservoir rule curve which establishes minimum reservoir levels at different times during the year, it will be possible to produce more energy in wetter years during winter than by following a set energy pattern. At the same time, the rule curve ensures that low flow sequences do not materially reduce the energy potential below a set minimum or firm annual energy.

The rule curves for Watana and Devil Canyon under combined operation are shown in Figure B.53.

4.3 - Operating Capabilities of Susitna Units

The operating conditions of both the Watana and Devil Canyon turbines are summarized in Table B.42.

(a) Watana

The Watana powerhouse will have six generating units with a nominal capacity of 170 MW corresponding to the minimum December reservoir level (Elevation 2117).

The gross head on the plant will vary from 590 feet to approximately 735 feet. The maximum unit output will change with head, as shown on Figure B.54.

The rated head for the turbine has been established at 680 feet, which is the weighted average operating head on the station. Allowing for generator losses, the rated turbine output is 250,000 hp (186.5 MW) at full gate.

The rated output of the turbines will be 250,000 hp at 680 feet rated net head. Maximum and minimum heads on the units will be 728 feet and 576 feet, respectively. The full gate output of the turbines will be about 275,000 hp at 728 feet net head and 195,000 hp at 576 feet net head. Overgating of the turbines may be possible, providing approximately 5 percent additional power; however, at high heads the turbine output will be restricted to avoid overloading the generators. The best efficiency point of the turbines will be established at the time of preparation of bid documents for the generating equipment and will be based on a detailed analysis of the anticipated operating range of the turbines. For preliminary design purposes, the best efficiency (best gate) output of the units has been assumed as 85 percent of the full gate turbine output. This percentage may vary from about 80 percent to 90 percent; in general, a lower percentage reduces turbine cost.

The full gate and best gate efficiencies of the turbines will be about 91 percent and 94 percent respectively at rated head. The efficiency will be about 0.5 percent lower at maximum head and 1 percent lower at minimum head. The preliminary performance curve for the turbine is shown on Figure B.55.

The Watana plant output may vary from zero, with the units at standstill or at spinning reserve, to approximately 1200 when all six units are operating under maximum output at maximum head. A graph of plant efficiency versus output and the number of on-line units is shown in Figure B.56. The load following requirements of the plant results in widely varying loading, but because of the multiple unit installation the total plant efficiency varies only slightly.

(b) Devil Canyon

The Devil Canyon powerhouse will have four generating units with a nominal capacity of 150 MW based on the minimum December reservoir level (Elevation 1405) and a corresponding gross head of 555 feet in the station.

The gross head on the plant will vary from 555 feet to 605 feet. The maximum unit output will change with head as shown in Figure B.57.

The rated average operating head for the turbine has been established at 575 feet. Allowing for generator losses, this results in a rated turbine output of 225,000 hp (168 MW) at full gate.

The generator rating has been selected as 180 MVA with a 90 percent power factor. The generators will be capable of continuous operation at 115 percent rated power. Because of the high capacity factor for the Devil Canyon station, the generators will therefore be sized on the basis of maximum turbine output at maximum head, allowing for a possible 5 percent addition in power from the turbine. This maximum turbine output (250,000 hp) is within the continuous overload rating of the generator.

Maximum and minimum heads on the units will be 542 feet and 600 feet, respectively. The full gate output of the turbines will be about 240,000 hp at maximum net head and 205,000 hp at minimum net head. Overgating of the turbines may be possible, providing approximately 5 percent additional power. For preliminary design purposes, the best efficiency (best gate) output of the units has been assumed at 85 percent of the full gate turbine output.

The full gate and best gate efficiencies of the turbines will be about 91 percent and 94 percent, respectively, at rated head. The efficiency will be about 0.2 percent lower at maximum head and 0.5 percent lower at minimum head. The preliminary performance curve for the turbine is shown in Figure B.58.

The Devil Canyon plant output may vary from zero to 700 MW with all four units operating at maximum output. The combined plant efficiency varies with output and number of units operating as shown in Figure B.59. As with Watana, the plant efficiency varies only slightly with loading due to the load following capabilities of multiple units.

4.4 - Tailwater Rating Curve

The tailwater rating curve for the Watana development is shown on Figure B.51 and for the Devil Canyon development on Figure B.52.

5 - STATEMENT OF POWER NEEDS AND UTILIZATION

5.1 - Railbelt Load Forecasts

In this section of the report, the electrical demand forecasts for the Railbelt region are described. Historical and projected trends are identified and discussed, and the forecasts used in Susitna generation planning studies are presented.

The feasibility of a major hydroelectric project depends in part upon the extent the available capacity and energy are consistent with the needs of the market to be served by the time the project comes on line. The Alaska Power Authority and the State of Alaska authorized load forecasts for the Alaska Railbelt region to be prepared independently of the Susitna feasibility study.

The Railbelt region, shown in Figure B.60, contains three electrical load centers: the Anchorage-Cook-Inlet area, the Fairbanks-Tanana Valley area, and the Glennallen-Valdez area. These areas are represented by the shaded areas in the figure. Because of the relatively small electrical requirements of the Glennallen-Valdez load center (approximately 2 percent of the demand of the Anchorage-Cook Inlet area) it is not specifically analyzed as an individual load center. For this study the Glennallen-Valdez load center is considered to be part of the Anchorage-Cook Inlet load center. The electrical demands for the Glennallen-Valdez area are determined as part of this study but are combined with the Anchorage-Cook Inlet loads. Future electrical requirements in excess of generating capacity are assumed to be served from the Anchorage area.

(a) Scope of Studies

There have been two forecasts developed and used during the feasibility study. In 1980, the Institute for Social and Economic Research (ISER) prepared economic and accompanying end use energy demand projections for the Railbelt. The end use forecasts were further refined as part of the feasibility study to estimate capacity demands and demand patterns. Also estimated was the potential impact on these forecasts of additional load management and energy conservation efforts. These forecasts were used in several portions of the feasibility study, including the development selection study, and initial economic, financial and sensitivity analyses. These forecasts are discussed in more detail in section (b) below.

In December 1981, Battelle Pacific Northwest Laboratories produced a series of revised load forecasts for the Railbelt. These forecasts were developed as a part of the Railbelt Alternatives Study, completed by Battelle under contract to the State of Alaska. Battelle's forecasts were a result of further updating of economic projections by ISER and some revised end-use models developed by Battelle, which took into account price sensitivity and several other factors not included in the 1980 projections. The December 1981 Battelle forecasts were used in the final project staging, economic, financial and sensitivity analyses. The December 1981 Battelle forecasts are presented in section (c) below.

(b) Electricity Demand Profiles

This section reviews the historical growth of electricity consumption in the Railbelt and compares it to the national trend. Earlier forecasts of Railbelt electricity consumption by ISER, which were used in Susitna development selection studies, are also described.

(i) Historical Trends

Between 1940 and 1978, electricity sales in the Railbelt grew at an average annual rate of 15.2 percent. This growth was roughly twice that for the nation as a whole. Table B.43 shows U.S. and Alaskan annual growth rates for different periods between 1940 and 1978. The historical growth of Railbelt utility sales from 1965 is illustrated in Figure B.61.

Although the Railbelt growth rates consistently exceeded the national average, the gap has been narrowing in later years due to the gradual maturing of the Alaskan economy. Growth in the Railbelt has exceeded the national average for two reasons: population growth in the Railbelt has been higher than the national rate, and the proportion of Alaskan households served by electric utilities was lower than the U.S. average so that some growth in the number of customers occurred independently of population growth. Table B.44 compares U.S. and Alaskan growth rates in the residential and commercial sectors.

The distribution of electricity consumption between residential and commercial-industrial-government sectors has been fairly stable. By 1978, the commercial-industrial-government and residential sectors accounted for 52 percent and 47 percent respectively. In contrast, the 1978 nationwide shares were 65 percent and 34 percent, respectively.

Historical electricity demand in the Railbelt, disaggregated by regions, is shown in Table B.45. During the period from 1965 to 1978, Greater Anchorage accounted for about 75 percent of Railbelt electricity consumption followed by Greater Fairbanks with 24 percent and Glennallen-Valdez with 1 percent. The pattern of regional sharing during this period has been quite stable and no discernible trend in regional shift has emerged. This is mainly a result of the uniform rate of economic development in the Alaskan Railbelt.

(ii) ISER Electricity Consumption Forecasts

The methodology used by ISER to estimate electric energy sales for the Railbelt is summarized in this section and the results obtained are discussed.

- Methodology

The ISER electricity demand forecasting model conceptualized in computer logic the linkage between economic growth scenarios and electricity consumption. The output from the model is in the form of projected values of electricity consumption for each of the three geographical areas of the Railbelt (Greater Anchorage, Greater Fairbanks and Glennallen-Valdez) and is classified by final use (i.e., heating, washing, cooling, etc.) and consuming sector (commercial, residential, etc). The model produces output on a five-year time basis from 1985 to 2010, inclusive.

The ISER model consists of several submodels linked by key variables and driven by policy and technical assumptions and state and national trends. These submodels are grouped into four economic models which forecast future levels of economic activity and four electricity consumption models which forecast the associated electricity requirements by consuming sectors. For two of the consuming sectors it was not possible to set up computer models and simplifying assumptions were made.

- Forecasting Uncertainty

To adequately address the uncertainty associated with the prediction of future demands, a number of different economic growth scenarios were considered. These were formulated by alternatively combining high, moderate and low growth rates in the area of special projects and industry with State government fiscal policies aimed at stimulating either high, moderate or low growth. This resulted in a total of nine potential growth scenarios for the state. In addition to these scenarios, ISER also considered the potential impact of a price reduced shift towards increased electricity demand. A short list of six future scenarios was selected. These concentrated around the mid-range or "base case" estimate of the upper and lower and extremes (see Table B.46).

- Demand Forecasts

An important factor to be considered in generation planning studies is the peak power demand associated with a forecast of electric energy demand. The overall approach to derivation of the peak demand forecasts for the Railbelt region was to examine the available historical data with regard to the generation of electrical energy and to apply the observed generation patterns to existing sales forecasts. Information routinely supplied by the Railbelt utilities to the Federal Energy Regulatory Commission was utilized to determine these load patterns.

The first step involved an adjustment to the allocated sales to reflect losses and energy unaccounted for. The adjustment was made by increasing the energy allocated to each utility by a factor computed from historical sales and generation levels. This resulted in a gross energy generation for each utility.

The factors determined for the monthly distribution of total annual generation were then used to distribute the gross generation for each year. The resulting hourly loads for each utility were added together to obtain the total Railbelt system load pattern for each forecast year. Table B.47 summarizes the total energy generation and the peak loads for each of the low, medium, and high ISER sales forecasts, assuming moderate government expenditure.

- Adjusted ISER Forecasts

Three of the initial ISER energy forecasts were considered in generation planning studies for development selection studies. These included the base case (MES-GM) or medium forecast, a low forecast and a high forecast. The low forecast was that corresponding to the low economic growth as proposed by ISER with an adjustment for low government expenditure (LES-GL). The high forecast corresponded to the ISER high economic growth scenario with an adjustment for high government expenditure (HES-GH).

The electricity forecasts summarized in Table B.47 represent total utility generation and include projections for self-supplied industrial and military generation sectors. Included in these forecasts are transmission and distribution losses in the range of 9 to 13 percent depending upon the generation scenario assumed. These forecasts, ranging from 2.71 to 4.76 percent average annual growth, were adjusted for use in generation planning studies.

The self-supplied industrial energy primarily involves drilling and offshore operations and other activities which are not likely to be connected into the Railbelt supply system. This component, which varies depending upon generation scenario, was therefore omitted from the forecasts used for planning purposes.

The military is likely to continue purchasing energy from the general market as long as it remains economic. However, much of their generating capacity is tied to district heating systems which would presumably continue operation. For study purposes, it was therefore assumed that 30 percent of the estimated military generation would be supplied from the grid system.

- The adjustments made to power and energy forecasts for use in self-supplied industrial and military sectors are reflected in Table B.47 and in Figure B.62. The power and energy values given in Table B.48 are those developed by ISER and used in the development selection studies. Annual growth rates range from 1.99 to 5.96 percent for very low and high forecasts with a medium generation forecast of 3.96 percent.

(c) Battelle Load Forecasts

As part of its study of Alaska Railbelt Electric Energy Alternatives, Reference 6, Battelle did extensive work in reviewing the 1980 ISER forecasts, methodology, and data, and produced a new series of forecasts. These forecasts built on the base of information and modeling established by ISER's 1980 work and, with the assistance of ISER, developed new models for forecasting Railbelt economic activity and resulting electrical energy demands. The resulting forecasts were adopted directly for use in final generation planning studies under this feasibility study.

These revised forecasts included both an energy and peak capacity projection for each year of the study period (1982-2010). The projection left out portions of electrical demand which would be self-supplied, such as much of the military demand and some of the industrial demand. In addition, these forecasts took into account the conservation technology and market penetration likely to take place. Details of the Battelle forecasts and methodology are available in a report produced by Battelle in early 1982 (8). The demand forecasting process is summarized in the following three paragraphs.

Figure B.63 shows the electricity demand forecasting process used by Battelle. The forecasting process contains two steps. The first step combines sets of consistent economic and policy assumptions (scenarios) with economic models from the ISER to produce forecasts of future economic activity, population, and households in the Railbelt region and its three load centers. In the second step, these forecasts are combined with data on current end uses of electricity in the residential sector, data on the size of the Railbelt commercial building stock, data on the cost and performance of conservation, assumptions concerning the future prices of electricity and other fuels, and future uses of electricity to produce demand forecasts.

The economic and population forecasts, energy use data, and other assumptions are all entered into a computer-based electricity demand forecasting model called the Railbelt Electricity Demand (RED) Model. The RED model generates forecasts of housing stock and commercial building stock and the price-adjusted intensity of energy use in both the residential and commercial (including government) sectors. It also adds estimates of major industrial electrical energy demand and miscellaneous uses such as street lighting. These forecasts are adjusted for specific energy conservation policies, and then the major end-use sector forecasts are combined by the model into forecasts of future annual demand for electric energy for each of the Railbelt's load centers. The combined annual loads are adjusted by an annual load factor to estimate future annual peak demand by load center. Finally, the peak loads are added together and multiplied by a diversity factor (to adjust for the fact that peak loads for different load centers do not coincide) to derive peak demand for the Railbelt. More detail on the REO model can be found in Reference 7.

The projected cost of power affects these forecasts. Because the size of demand for power affects the size, number, and cost of generating facilities that may have to be built to meet the demand (which in turn affects the cost of power), several passes through the REO model with constant economic assumptions and varying costs of power are required to produce a final forecast.

The Battelle study produced numerous load forecasts which corresponded to different development plans. The plans varied due to different economic scenarios and costs of power. From these separate forecasts, a high, medium and low forecast were selected for project planning and economic and financial feasibility studies.

The Battelle forecasts are based on energy sales, and have therefore been adjusted by an addition of an estimated 8 percent for transmission losses to arrive at the supply forecast to be used in generation planning. Table B.49 and Figure B.64 present the three Battelle forecasts which were prepared to bracket the range of electrical demand for the future.

It should be noted that the load forecast figures vary in absolute values of peak demand and energy from those figures in the referenced Battelle studies. This minor variance (approximately 5-8 percent in the project development years) is due to the revision in the Battelle forecasts in 1982 after the feasibility work on Susitna proceeded using December 1981 numbers.

The Battelle forecasts were used in second stage generation planning studies. The second stage studies focused on the economic and financial feasibility of the selected Susitna project and the sensitivity of the analyses to variation of key study assumptions. The differences between the earlier ISER forecasts used in development selection studies and the revised Battelle forecasts are not considered to be significant enough to have altered the conclusions of the earlier studies. The Railbelt generation planning studies undertaken for Susitna feasibility assessment were based on the Battelle medium forecast. The high and low Battelle forecasts were used as a basis for sensitivity testing.

No additional information on load patterns relative to monthly and daily shifting of load shapes was developed in the Battelle forecasts. Thus, the historical data developed to use with the 1980 ISER forecasts were also used with the Battelle forecasts.

5.2 - Market and Price for Watana Output in 1994

It has been planned that Watana energy will be supplied at a single wholesale rate on a free market basis. This requires, in effect, that Susitna energy be priced so that it is attractive even to utilities with the lowest cost alternative source of energy. On this basis it is estimated that for the marketable 3315 GWh of energy generated by Watana in 1994 to be attractive, a price of 145 mills per kWh in 1994 dollars is required. Justification for this price is illustrated in Figure B.65. Note that the assumption is made that the only capital costs which would be avoided in the early 1990s would be those due to the addition of new coal-fired generating plants (i.e., the alternative 2 x 200 MW coal-fired Beluga station).

The financing considerations under which it would be appropriate for Watana energy to be sold at approximately 145 mills/kWh price are presented in Exhibit D; however, it should be noted that some of the energy which would be displaced by Watana's 3315 GWh would have been generated at a lower cost than 145 mills, and utilities might wish to delay accepting it at this price until the escalating cost of natural gas or other fuels made it more attractive. A number of approaches to the resolution of this problem can be postulated, including pre-contract arrangements.

(a) Contractual Preconditions for Susitna Energy Sale

It will be necessary to contract with Railbelt Utilities for the purchase of Susitna capacity and energy on a basis appropriate to support financing of the project.

Pricing policies for Susitna output are assumed to be constrained by both cost (as defined by State of Alaska Senate Bill 25) and by the price of energy from the best thermal option.

Marketing Susitna's output within these twin constraints would ensure that all state support for Susitna flowed through to consumers and under no circumstances were prices to consumers higher than they would have been under the best thermal option. In addition, consumers would also obtain the long-term economic benefits of Susitna's low cost energy.

(b) Market Price for Watana Output 1995-2001

After its initial entry into the system in 1994, the price and market for the 3315 Gwh of Watana output is consistently upheld over the years to 2001 by the projected 20 percent increase in total demand over this period.

There would, as a result, be a 70 percent increase in cost savings compared with the best thermal alternative. The increasing cost per unit of output from a system without Susitna is illustrated in Figure B.66.

(c) Market and Price for Watana and Devil Canyon Output in 2003

A diagramatic analysis of the total cost savings which the combined Watana and Devil Canyon output will confer on the system compared with the present thermal option in the year 2003 is shown in Figure B.67. These total savings are divided by the energy contributed by Susitna to indicate a price of 250 mills per kWh as the maximum price which can be charged for Susitna output. Here again, the problem of competing with lower cost combined cycle, gas turbines, etc., will have to be addressed; however, this problem is likely to be short term in nature, since by this time period these thermal power facilities will be approaching retirement.

Only about 90 percent of the total Susitna output will be absorbed by the system in 2002; the balance of the output will be progressively absorbed over the following decade. This will provide increasing total savings to the system from Susitna with no associated increase in costs.

(d) Potential Impact of State Appropriations

In the preceding paragraphs the maximum price at which Susitna energy could be sold has been identified. Sale of the energy at these prices will depend upon the magnitude of any proposed state appropriation designed to reduce the cost of Susitna energy in the earlier years. At significantly lower prices it is likely that the total system demand will be higher than assumed. This, combined with a state appropriation to reduce the energy cost of Watana energy, would make it correspondingly easier to market the output from the Susitna development; however, as the preceding analysis shows, a viable and strengthening market exists for the energy from the development that would make it possible to price the output up to the cost of the best thermal alternative.

(e) Conclusions

Based on the assessment of the market for power and energy output from the Susitna Hydroelectric Project, it has been concluded that, with the appropriate level of state appropriation and with pricing as defined in Senate Bill 25, an attractive basis exists, particularly in the long term, for the Railbelt utilities to derive benefit from the project. It should be recognized that contractual arrangements covering purchase of Susitna output will be an essential precondition for the actual commencement of project construction. These contractual arrangements will be pursued during the licensing and design phase of the project.

5.3 - Sale of Power

Electrical energy from the Susitna Hydroelectric Project will be sold to utilities serving the Anchorage/Fairbanks net.

The potential customers for Susitna power utilities in the Railbelt include:

- Fairbanks Municipal Utility System;
- Homer Electric Association;
- Anchorage Municipal Light & Power Department;
- Chugach Electric Association;
- Golden Valley Electric Association;
- Matanuska Electric Association;
- Seward Electric System; and
- Copper Valley Electric Association.

A more detailed discussion of marketing can be found in Reference 8.

6 - FUTURE SUSITNA BASIN DEVELOPMENT

The Alaska Power Authority has no current plans for further development of the Watana/Devil Canyon system and no plans for further water power projects in the Susitna River Basin at this time.

Development of the proposed projects would preclude further major hydroelectric development in the Susitna basin, with the exception of major storage projects in the Susitna basin headwaters. Although these type of plans have been considered in the past, they are neither active nor anticipated to be so in the foreseeable future.

EXHIBIT B - STATEMENT OF PROJECT OPERATION AND RESOURCES UTILIZATION

LIST OF REFERENCES

1. Acres American Inc., Susitna Hydroelectric Project, Development Selection Report, prepared for the Alaska Power Authority, December 1981.

2. Woodward-Clyde Consultants, Final Report on Seismic Studies for Susitna Hydroelectric Project, prepared for Acres American Inc., February 1982.

3. Acres American Inc., Susitna Hydroelectric Project, 1980-81 Geotechnical Report, prepared for the Alaska Power Authority, February 1982.

4. Acres American Inc., Susitna Hydroelectric Project, Feasibility Report, prepared for the Alaska Power Authority, March 1982.

5. General Electric Company, OGP5 User's Manual, May 1979.

6. Battelle Pacific Northwest Laboratories, Railbelt Electric Power Alternatives Study: Evaluation of Railbelt Electric Energy Plans, prepared for the Office of the Governor, State of Alaska, August 1982.

7. Battelle Pacific Northwest Laboratories, The Railbelt Electricity Demand (RED) Model Specifications Report, prepared for the Office of the Governor, State of Alaska, August 1982.

8. Acres American Inc., Susitna Hydroelectric Project Reference Report, Economic, Marketing and Financial Evaluation, prepared for the Alaska Power Authority, April 1982.

TABLE B.1: POTENTIAL HYDROELECTRIC DEVELOPMENT

Site	Dam Proposed Type	Dam Height Ft.	Upstream Regulation	Capital Cost $ million (1980)	Installed Capacity (MW)	Average Annual Energy Gwh	Economic Cost of Energy $/1000 kWh[1]	Source of Data
Gold Creek[2]	Fill	190	Yes	900	260	1,140	37	USBR 1953
Olson (Susitna II)	Concrete	160	Yes	600	200	915	31	USBR 1953 KAISER 1974 COE 1975
Devil Canyon	Concrete	675	No Yes	830 1,000	250 600	1,420 2,980	27 17	This Study "
High Devil Canyon (Susitna I)	Fill	855	No	1,500	800	3,540	21	" "
Devil Creek[2]	Fill	Approx 850	No	-	-	-	-	"
Watana	Fill	880	No	1,860	800	3,250	28	"
Susitna III	Fill	670	No	1,390	350	1,580	41	"
Vee	Fill	610	No	1,060	400	1,370	37	"
Maclaren[2]	Fill	185	No	530[4]	55	180	124	"
Denali	Fill	230	No	480[4]	60	245	81	"
Butte Creek[2]	Fill	Approx 150	No	-	40	130[3]	-	USBR 1953
Tyone[2]	Fill	Approx 60	No	-	6	22[3]	-	USBR 1953

Notes:

(1) Includes AFDC, Insurance, Amortization, and Operation and Maintenance Costs.
(2) No detailed engineering or energy studies undertaken as part of this study.
(3) These are approximate estimates and serve only to represent the potential of these two damsites in perspective.
(4) Include estimated costs of power generation facility.

TABLE B.2 - COST COMPARISONS

DAM		Capital Cost Estimate[2] (1980 $)				
		ACRES 1980		OTHERS		
Site	Type	Installed Capacity - MW	Capital Cost $ million	Installed Capacity - MW	Capital Cost $ million	Source and Date of Data
Gold Creek	Fill	-	-	260[1]	890	USRB 1968
Olson (Susitna II)	Concrete	-	-	190[1]	550	COE 1975
Devil Canyon	Fill	600	1,000	-	-	-
	Concrete Arch	-	-	776	630	COE 1975
	Concrete Gravity	-	-	776	910	COE 1978
High Devil Canyon (Susitna I)	Fill	800	1,500	700	1,480	COE 1975
Devil Creek	Fill	-	-	-	-	-
Watana	Fill	800	1,860	792	1,630	COE 1978
Susitna III	Fill	350	1,390	445	-	KAISER 1974
Vee	Fill	400	1,060	-	770	COE 1975
Maclaren	Fill	55	530	-		-
Denali	Fill	60	480	None	500	COE 1975

Notes:

(1) Dependable Capacity
(2) Excluding Anchorage/Fairbanks transmission intertie, but including local access and transmission.

TABLE B.3: DAM CREST AND FULL SUPPLY LEVELS

Site	Staged Dam Construction	Full Supply Level - Ft.	Dam Crest Level - Ft.	Average Tailwater Level - ft.	Dam Height[1] ft.
Gold Creek	No	870	880	680	290
Olson	No	1,020	1,030	810	310
Portage Creek	No	1,020	1,030	870	250
Devil Canyon - intermediate height	No	1,250	1,270	890	465
Devil Canyon - full height	No	1,450	1,470	890	675
High Devil Canyon	No	1,610	1,630	1,030	710
	No	1,750	1,775	1,030	855
Watana	Yes	2,000	2,060	1,465	680
	Stage 2	2,200	2,225	1,465	880
Susitna III	No	2,340	2,360	1,810	670
Vee	No	2,330	2,350	1,925	610
Maclaren	No	2,395	2,405	2,300	185
Denali	No	2,540	2,555	2,405	230

Notes:

(1) To foundation level.

TABLE B.4 - CAPITAL COST ESTIMATE SUMMARIES
SUSITNA BASIN DAM SCHEMES
COST IN $MILLION 1980

Item	Devil Canyon 1470 ft Crest 600 MW	High Devil Canyon 1775 ft Crest 800 MW	Watana 2225 ft Crest 800 MW	Susitna III 2360 ft Crest 330 MW	Vee 2350 ft Crest 400 MW	Maclaren 2405 ft Crest No power	Denali 2250 ft Crest No power
1) Lands, Damages & Reservoirs	26	11	46	13	22	25	38
2) Diversion Works	50	48	71	88	37	118	112
3) Main Dam	166	432	536	398	183	106	100
4) Auxiliary Dam	0	0	0	0	40	0	0
5) Power System	195	232	244	140	175	0	0
6) Spillway System	130	141	165	121	74	0	0
7) Roads and Bridges	45	68	96	70	80	57	14
8) Transmission Line	10	10	26	40	49	0	0
9) Camp Facilities and Support	97	140	160	130	100	53	50
10) Miscellaneous[1]	8	8	8	8	8	5	5
11) Mobilization and Preparation	30	47	57	45	35	15	14
Subtotal	757	1137	1409	1053	803	379	333
Contingency (20%)	152	227	282	211	161	76	67
Engineering and Owner's Administration (12%)	91	136	169	126	96	45	40
TOTAL	1000	1500	1860	1390	1060	500	440

Notes:

(1) Includes recreational facilities, buildings and grounds and permanent operating equipment.

TABLE B.5 - RESULTS OF SCREENING MODEL

	Total Demand		Optimal Solution				First Suboptimal Solution				Second Suboptimal Soultion			
Run	Cap. MW	Energy GWh	Site Names	Max. Water Level	Inst. Cap. MW	Total Cost $ million	Site Names	Max. Water Level	Inst. Cap. MW	Total Cost $ million	Site Names	Max. Water Level	Inst. Cap. MW	Total Cost $ million
1	400	1750	High Devil Canyon	1580	400	885	Devil Canyon	1450	400	970	Watana	1950	400	980
2	800	3500	High Devil Canyon	1750	800	1500	Watana	1900	450	1130	Watana	2200	800	1860
							Devil Canyon	1250	350	710				
							TOTAL		800	1840				
3	1200	5250	Watana	2110	700	1690	High Devil Canyon	1750	800	1500	High Devil Canyon	1750	820	1500
			Devil Canyon	1350	500	800	Vee	2350	400	1060	Susitna III	2[illegible]00	380	1260
			TOTAL		1200	2490	TOTAL		1200	2560	TOTAL		1200	2760
4	1400	6150	Watana	2150	740	1770	NO SOLUTION				NO SOLUTION			
			Devil Canyon	1450	660	1000								

TABLE B.6: INFORMATION ON THE DEVIL CANYON DAM AND TUNNEL SCHEMES

Item	Devil Canyon Dam	Tunnel Scheme 1	Tunnel Scheme 2	Tunnel Scheme 3	Tunnel Scheme 4
Reservoir Area (Acres)	7,500	320	0	3,900	0
River Miles Flooded	31.6	2.0	0	15.8	0
Tunnel Length (Miles)	0	27	29	13.5	29
Tunnel Volume (1000 Yd^3)	0	11,976	12,863	3,732	5,131
Compensating Flow Release (cfs)	0	1,000	1,000	1,000	1,000
Reservoir Volume (1000 Acre-feet)	1,100	9.5	--	350	--
Dam Height (feet)	625	75	--	245	--
Typical Daily Range of Discharge From Devil Canyon Powerhouse (cfs)	6,000 to 13,000	4,000 to 14,000	4,000 to 14,000	8,300 to 8,900	3,900 to 4,200
Approximate Maximum Daily Fluctuations in Reservoir (feet)	2	15	--	4	--

Notes:

[3] Estimated, above existing rock elevation.

TABLE B.7 - DEVIL CANYON TUNNEL SCHEMES
COSTS, POWER OUTPUT AND AVERAGE ANNUAL ENERGY

Stage	Installed Capacity (MW) Watana	Installed Capacity (MW) Devil Canyon Tunnel	Increase[1] in Installed Capacity (MW)	Devil Canyon Average Annual Energy (Gwh)	Increase[1] in Average Annual Energy (Gwh)	Tunnel Scheme Total Project Costs $ Million	Cost[3] of Additional Energy[1] (mills/kWh)
STAGE 1:							
Watana Dam	800	---	---	---	---	---	---
STAGE 2:							
Tunnel:							
- Scheme 1	800	550	550	2,050	2,050	1980	42.6
- Scheme 2	70	1,150	420	4,750	1,900	2320	52.9
- Scheme 3[2]	850	330	380	2,240	2,180	1220	24.9
- Scheme 4	800	365	365	2,490	890	1490	73.6

Notes:

(1) Increase over single Watana, 800 MW development 3250 Gwh/yr
(2) Includes power and energy produced at re-regulation dam
(3) Energy cost is based on an economic analysis (i.e. using 3 percent interest rate)

TABLE B.8 - CAPITAL COST ESTIMATE SUMMARIES
TUNNEL SCHEMES
COSTS IN $MILLION 1980

Item		Two 30 ft dia tunnels		One 40 ft dia tunnel
Land and damages, reservoir clearing		14		14
Diversion works		35		35
Re-regulation dam		102		102
Power system		680		576
(a) Main tunnels	557		453	
(b) Intake, powerhouse, tailrace and switchyard	123		123	
Secondary power station		21		21
Spillway system		42		42
Roads and bridges		42		42
Transmission lines		15		15
Camp facilities and support		131		117
Miscellaneous*		8		8
Mobilization and preparation		47		47
TOTAL CONSTRUCTION COST		1,137		1,015
Contingencies (20%)		227		203
Engineering, and Owner's Administration		136		122
TOTAL PROJECT COST		1,500		1,340

TABLE B.9. SUSITNA DEVELOPMENT PLANS

			Stage/Incremental Data				Cumulative System Data		
Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft	Annual Energy Production Firm GWH	Avg. GWH.	Plant Factor %
1.1	1	Watana 2225 ft 800MW	1860	1993	2200	150	2670	3250	46
	2	Devil Canyon 1470 ft 600 MW	1000	1996	1450	100	5500	6230	51
		TOTAL SYSTEM 1400 MW	2860						
1.2	1	Watana 2060 ft 400 MW	1570	1992	2000	100	1710	2110	60
	2	Watana raise to 2225 ft	360	1995	2200	150	2670	2990	85
	3	Watana add 400 MW capacity	130[2]	1995	2200	150	2670	3250	46
	4	Devil Canyon 1470 ft 600 MW	1000	1996	1450	100	5500	6230	51
		TOTAL SYSTEM 1400 MW	3060						
1.3	1	Watana 2225 ft 400 MW	1740	1993	2200	150	2670	2990	85
	2	Watana add 400 MW capacity	150	1993	2200	150	2670	3250	46
	3	Devil Canyon 1470 ft 600 MW	1000	1996	1450	100	5500	6230	51
		TOTAL SYSTEM 1400 MW	2890						

TABLE B.9 (Continued)

Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft.	Annual Energy Production Firm GWH	Annual Energy Production Avg. GWH	Plant Factor %
			Stage/Incremental Data				Cumulative System Data		
2.1	1	High Devil Canyon							
		1775 ft 800 MW	1500	1994[3]	1750	150	2460	3400	49
	2	Vee 2350 ft 400 MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200 MW	2560						
2.2	1	High Devil Canyon							
		1630 ft 400 MW	1140	1993[3]	1610	100	1770	2020	58
	2	High Devil Canyon add 400 MW Capacity raise dam to 1775 ft	500	1996	1750	150	2460	3400	49
	3	Vee 2350 ft 400 MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200 MW	2700						
2.3	1	High Devil Canyon							
		1775 ft 400 MW	1390	1994[3]	1750	150	2400	2760	79
	2	High Devil Canyon add 400 MW capacity	140	1994	1750	150	2460	3400	49
	3	Vee 2350 ft 400 MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200 MW	2590						
3.1	1	Watana 2225 ft 800 MW	1860	1993	2200	150	2670	3250	46
	2	Watana add [illegible]0 MW tunnel 330 MW	1500	1995	1475	4	4890	5430	53
		TOTAL SYSTEM 1180 MW	3360						

TABLE B.9 (Continued)

			Stage/Incremental Data				Cumulative System Data		
Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft.	Annual Energy Production Firm GWH	Annual Energy Production Avg. GWH	Plant Factor %
3.2	1	Watana 2225 ft 400 MW	1740	1993	2200	150	2670	2990	85
	2	Watana add 400 MW capacity	150	1994	2200	150	2670	3250	46
	3	Tunnel 330 MW add 50 MW to Watana	1500	1995	1475	4	4890	5430	53
			3390						
4.1	1	Watana 2225 ft 400 MW	1740	1995[3]	2200	150	2670	2990	85
	2	Watana add 400 MW capacity	150	1996	2200	150	2670	3250	46
	3	High Devil Canyon 1470 ft 400 MW	860	1998	1450	100	4520	5280	50
	4	Portage Creek 1030 ft 150 MW	650	2000	1020	50	5110	6000	51
		TOTAL SYSTEM 1350 MW	3400						

NOTES:

(1) Allowing for a 3 year overlap construction period between major dams.
(2) Plan 1.2 Stage 3 is less expensive than Plan 1.3 Stage 2 due to lower mobilization costs.
(3) Assumes FERC license can be filed by June 1984, ie. 2 years later than for the Watana/Devil Canyon Plan 1.

TABLE B.10. SUSITNA ENVIRONMENTAL DEVELOPMENT PLANS

Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft	Annual Energy Production Firm GWH	Annual Energy Production Avg. GWH.	Plant Factor %
			Stage/Incremental Data				Cumulative System Data		
E1.1	1	Watana 2225 ft 800MW and Re-Regulation Dam	1960	1993	2200	150	2670	3250	46
	2	Devil Canyon 1470 ft 400MW	900	1996	1450	100	5520	6070	58
		TOTAL SYSTEM 1200MW	2860						
E1.2	1	Watana 2060 ft 400MW	1570	1992	2000	100	1710	2110	60
	2	Watana raise to 2225 ft	360	1995	2200	150	2670	2990	85
	3	Watana add 400MW capacity and Re-Regulation Dam	230[2]	1995	2200	150	2670	3250	46
	4	Devil Canyon 1470 ft 400MW	900	1996	1450	100	5520	6070	58
		TOTAL SYSTEM 1200MW	3060						
E1.3	1	Watana 2225 ft 400MW	1740	1993	2200	150	2670	2990	85
	2	Watana add 400MW capacity and Re-Regulation Dam	250	1993	2200	150	2670	3250	46
	3	Devil Canyon 1470 ft 400 MW	900	1996	1450	100	5520	6070	58
		TOTAL SYSTEM 1200MW	2890						

TABLE B.10 (Continued)

			Stage/Incremental Data				Cumulative System Data		
Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft.	Annual Energy Production Firm GWH	Avg. GWH	Plant Factor %
E1.4	1	Watana 2225 ft 400MW	1740	1993	2200	150	2670	2990	85
	2	Devil Canyon 1470 ft 400MW	900	1996	1450	100	5190	5670	81
		TOTAL SYSTEM 800MW	2640						
E2.1	1	High Devil Canyon 1775 ft 800MW and Re-Regulation Dam	1600	1994[3]	1750	150	2460	3400	49
	2	Vee 2350ft 400MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200MW	2660						
E2.2	1	High Devil Canyon 1630 ft 400MW	1140	1993[3]	1610	100	1770	2020	58
	2	High Devil Canyon raise dam to 1775 ft add 400MW and Re-Regulation Dam	600	1996	1750	150	2460	3400	49
	3	Vee 2350 ft 400 MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200MW	2800						
E2.3	1	High Devil Canyon 1775 ft 400MW	1390	1994[3]	1750	150	2400	2760	79
	2	High Devil Canyon add 400MW capacity and Re-Regulation Dam	240	1995	1750	150	2460	3400	49
	3	Vee 2350 ft 400MW	1060	1997	2330	150	3870	4910	47
		TOTAL SYSTEM 1200	2690						

TABLE B.10 (Continued)

Plan	Stage	Construction	Capital Cost $ Millions (1980 values)	Earliest On-line Date[1]	Reservoir Full Supply Level - ft.	Maximum Seasonal Draw-down-ft.	Annual Energy Production Firm GWH	Annual Energy Production Avg. GWH	Plant Factor %
			Stage/Incremental Data				Cumulative System Data		
E2.4	1	High Devil Canyon 1755 ft 400MW	1390	1994[3]	1750	150	2400	2760	79
	2	High Devil Canyon add 400MW capacity and Portage Creek Dam 150 ft	790	1995	1750	150	3170	4080	49
	3	Vee 2350 ft 400MW	1060	1997	2330	150	4430	5540	47
		TOTAL SYSTEM	3240						
E3.2	1	Watana 2225 ft 400MW	1740	1993	2200	150	2670	2990	85
	2	Watana add 400 MW capacity and Re-Regulation Dam	250	1994	2200	150	2670	3250	46
	3	Watana add 50MW Tunnel Scheme 330MW	1500	1995	1475	4	4890	5430	53
		TOTAL SYSTEM 1180MW	3490						
E4.1	1	Watana 2225 ft 400MW	1740	1995[3]	2200	150	2670	2990	85
	2	Watana add 400MW capacity and Re-Regulation Dam	250	1996	2200	150	2670	3250	46
	3	High Devil Canyon 1470 ft 400MW	860	1998	1450	100	4520	5280	50
	4	Portage Creek 1030 ft 150MW	650	2000	1020	50	5110	6000	51
		TOTAL SYSTEM 1350 MW	3500						

NOTES:
(1) Allowing for a 3 year overlap construction period between major dams.
(2) Plan 1.2 Stage 3 is less expensive than Plan 1.3 Stage 2 due to lower mobilization costs.
(3) Assumes FERC license can be filed by June 1984, ie. 2 years later than for the Watana/Devil Canyon Plan 1.

TABLE B.11 - RESULTS OF ECONOMIC ANALYSES OF SUSITNA PLANS - MEDIUM LOAD FORECAST

Susitna Development Plan Inc.						Installed Capacity (MW) by Category in 2010							
	Online Dates Stages					Thermal			Hydro				
Plan No.	1	2	3	4	OGP5 Run Id. No.	Coal	Gas	Oil	Other	Susitna	Total System Installed Capacity In 2010-MW	Total System Present Worth Cost $ Million[1]	Remarks Pertaining to the Susitna Basin Development Plan
E1.1	1993	2000	--	--	LXE7	300	426	0	144	1200	2070	5850	
E1.2	1992	1995	1997	2002	L5Y9	200	501	0	144	1200	2045	6030	
E1.3	1993	1996	2000	--	L8J9	300	426	0	144	1200	2070	5850	
	1993	1996	--	--	L7W7	500	651	0	144	800	2095	6960	Stage 3, Devil Canyon Dam not constructed
	1998	2001	2005	--	LAD7	400	276	30	144	1200	2050	6070	Delayed implementation schedule
E1.4	1993	2000	--	--	LCK5	200	726	50	144	800	1920	5890	Total development limited to 800 MW
Modified E2.1	1994	2000	--	--	LB25	400	651	60	144	800	2055	6620	High Devil Canyon limited to 400 MW
E2.3[1]	1993	1996	2000	--	L601	300	651	20	144	1200	2315	6370	
	1993	1996	--	--	LE07	500	651	30	144	800	2125	6720	Stage 3, Vee Dam, not constructed
Modified E2.3	1993	1996	2000		LEB3	300	726	220	144	1300	2690	6210	Vee dam replaced by Chakachamna dam
3.1	1993	1996	2000	--	L607	200	651	30	144	1180	2205	6530	
Special 3.1	1993	1996	2000	--	L615	200	651	30	144	1180	2205	6230	Capital cost of tunnel reduced by 50 percent
E4.1	1995	1996	1998	--	LTZ5	200	576	30	144	1200	2150	6050	Stage 4 not constructed

NOTES:

(1) Adjusted to incorporate cost of re-regulation dam

TABLE B.12 - RESULTS OF ECONOMIC ANALYSES OF SUSITNA PLANS - LOW AND HIGH LOAD FORECAST

Plan No.	Susitna Development Plan Inc. Online Dates Stages 1	2	3	4	OGP5 Run Id. No.	Installed Capacity (MW) by Category in 2010: Thermal Coal	Thermal Gas	Thermal Oil	Hydro Other	Hydro Susitna	Total System Installed Capacity In 2010-MW	Total System Present Worth Cost $ Million	Remarks Pertaining to the Susitna Basin Development Plan
VERY LOW FORECAST[1]													
E1.4	1997	2005	--	--	L7B7	0	651	50	144	800	1645	3650	
LOW LOAD FORECAST													
E1.3	1993	1996	2000	--	--	--	--	--	--	--	--	--	Low energy demand does not warrant plan capacities
E1.4	1993	2002	--	--	LC07	0	351	40	144	800	1335	4350	
	1993	--	--	--	LBK7	200	501	80	144	400	1325	4940	Stage 2, Devil Canyon Dam, not constructed
E2.1	1993	2002	--	--	LG09	100	426	30	144	800	1500	4560	High Devil Canyon limited to 400 MW
	1993	--	--	--	LBU1	400	501	0	144	400	1445	4850	Stage 2, Vee Dam, not constructed
E2.3	1993	1996	2000	--	--	--	--	--	--	--	--	--	Low energy demand does not warrant plan capacities
Special 3.1	1993	1996	2000	--	L613	0	576	20	144	780	1520	4730	Capital cost of tunnel reduced by 50 percent
3.2	1993	2002	--	--	L609	0	576	20	144	780	1520	5000	Stage 2, 400 MW addition to Watana, not constructed
HIGH LOAD FORECAST													
E1.3	1993	1996	2000	--	LA73	1000	951	0	144	1200	3295	10680	
Modified E1.3	1993	1996	2000	2005[2]	LBV7	800	651	60	144	1700	3355	10050	Chakachamna hydroelectric generating station (480 MW) brought on line as a fourth stage
E2.3	1993	1996	2000	--	LBV3	1300	951	90	144	1200	3685	11720	
Modified E2.3	1993	1996	2000	2003[2]	LBY1	1000	876	10	144	1700	3730	11040	Chakachamna hydroelectric generating station (480 MW) brought on line as a fourth stage

NOTE:

(1) Incorporating load management and conservation

TABLE B.13 - ANNUAL FIXED CARRYING CHARGES

Project Type		Economic Life - Years	Cost of Money %	Amortization %	Insurance %
			Economic Parameters		
Thermal	- Gas Turbine (Oil Fired)	20	3.00	3.72	0.25
	- Diesel, Gas Turbine (Gas Fired) and Large Steam Turbine	30	3.00	2.10	0.25
	- Small Steam Turbine	35	3.00	1.65	0.25
Hydropower		50	3.00	0.89	0.10

FUEL COSTS AND ESCALATION RATES

	Natural Gas	Coal	Distillate
Base Period (January 1980)			
- Prices ($/million Btu)			
Market Prices	$1.05	$1.15	$4.00
Shadow (Opportunity) Values	2.00	1.15	4.00
Real Escalation Rates (Percentage)			
- Change Compounded (Annually)			
1980 - 1985	1.79%	9.56%	3.38%
1986 - 1990	6.20	2.39	3.09
1991 - 1995	3.99	-2.87	4.27
Composite (average) 1980-1995	3.98	2.93	3.58
1996 - 2005	3.98	2.93	3.58
2006 - 2010	0	0	0

TABLE B.14 - SUMMARY OF THERMAL GENERATING RESOURCE PLANT PARAMETERS

Parameter	PLANT TYPE					
	COAL-FIRED STEAM			COMBINED CYCLE	GAS TURBINE	DIESEL
	500 MW	250 MW	100 MW	250 MW	75 MW	10 MW
Heat Rate (Btu/kWh)	10,500	10,500	10,500	8,500	12,000	11,500
O&M Costs						
Fixed O&M ($/yr/kW)	0.50	1.05	1.30	2.75	2.75	0.50
Variable O&M ($/MWH)	1.40	1.80	2.20	0.30	0.30	5.00
Outages						
Planned Outages (%)	11	11	11	14	11	1
Forced Outages (%)	5	5	5	6	3.8	5
Construction Period (yrs)	6	6	5	3	2	1
Start-up Time (yrs)	6	6	6	4	4	1
Total Capital Cost ($ million)						
Railbelt:	-	-	-	175	26	7.7
Beluga:	1,130	630	290	-	-	-
Unit Capital Cost ($/kW)[1]						
Railbelt:	-	-	-	728	250	778
Beluga:	2473	2744	3102	-	-	-

Notes:

(1) Including AFDC at 0 percent escalation and 3 percent interest.

TABLE B.15 - ECONOMIC BACKUP DATA FOR EVALUATION OF PLANS

Parameter	Total Present Worth Cost for 1981 - 2040 Period $ Million (% Total) Generation Plan With High Devil Canyon - Vee	Generation Plan With Watana - Devil Canyon Dam	Generation Plan With Watana - Tunnel	All Thermal Generation Plans
Capital Investment	2800 (44)	2740 (47)	3170 (49)	2520 (31)
Fuel	3220 (50)	2780 (47)	3020 (46)	5240 (64)
Operation and Maintenance	350 (6)	330 (6)	340 (5)	370 (5)
TOTAL:	6370 (100)	5850 (100)	6530 (100)	8130 (100)

TABLE B.16 - ECONOMIC EVALUATION OF DEVIL CANYON DAM AND TUNNEL SCHEMES AND WATANA/DEVIL CANYON AND HIGH DEVIL CANYON/VEE PLANS

		Present worth of Net Benefit ($ million) of total generation system costs for the: Devil Canyon Dam over the Tunnel Scheme	Watana/Devil Canyon Dams over the High Devil Canyon/Vee Dams	Remarks
ECONOMIC EVALUATION:				
- Base Case		680	520	Economic ranking: Devil Canyon dam scheme is superior to Tunnel scheme. Watana/Devil Canyon dam plan is superior to the High Devil Canyon dam/Vee dam plan.
SENSITIVITY ANALYSES:				
- Load Growth	Low High	650 N.A.	210 1040	The net benefit of the Watana/Devil Canyon plan remains positive for the range of load forecasts considered. No change in ranking.
- Capital Cost Estimate		Higher uncertainty associated with tunnel scheme.	Higher uncertainty associated with H.D.C./Vee plan.	Higher cost uncertainties associated with higher cost schemes/plans. Cost uncertainty therefore does not affect economic ranking.
- Period of Economic Analysis	Period shortened to (1980 - 2010)	230	160	Shorter period of evaluation decreases economic differences. Ranking remains unchanged.
- Discount Rate	5% 8% (interpolated) 9%	As both the capital and fuel costs associated with the tunnel scheme and H.D.C./Vee Plan are higher than for Watana/Devil Canyon plan any changes to these parameters cannot reduce the Devil Canyon or Watana/Devil Canyon net benefit to below zero.		Ranking remains unchanged.
- Fuel Cost	80% basic fuel cost			
- Fuel Cost Escalation	0% fuel escalation 0% coal escalation			
- Economic Thermal Plant Life	50% extension 0% extension			

TABLE B.17 - ENVIRONMENTAL EVALUATION OF DEVIL CANYON DAM AND TUNNEL SCHEME

Environmental Attribute	Concerns	Appraisal (Differences in impact of two schemes)	Identification of difference	Appraisal Judgement	Scheme judged to have the least potential impact: Tunnel	Scheme judged to have the least potential impact: DC
Ecological:						
- Downstream Fisheries and Wildlife	Effects resulting from changes in water quantity and quality.	No significant difference between schemes regarding effects downstream of Devil Canyon.	---	Not a factor in evaluation of scheme.		
		Difference in reach between Devil Canyon dam and tunnel re-regulation dam.	With the tunnel scheme controlled flows between regulation dam and downstream powerhouse offers potential for anadromous fisheries enhancement in this 11 mile reach of the river.	If fisheries enhancement opportunity can be realized the tunnel scheme offers a positive mitigation measure not available with the Devil Canyon dam scheme. This opportunity is considered moderate and favors the tunnel scheme. However, there are no current plans for such enhancement and feasibility is uncertain. Potential value is therefore not significant relative to additional cost of tunnel.	X	
Resident Fisheries:	Loss of resident fisheries habitat.	Minimal differences between schemes.	Devil Canyon dam would inundate 27 miles of the Susitna River and approximately 2 miles of Devil Creek. The tunnel scheme would inundate 16 miles of the Susitna River.	Loss of habitat with dam scheme is less than 5% of total for Susitna main stem. This reach of river is therefore not considered to be highly significant for resident fisheries and thus the difference between the schemes is minor and favors the tunnel scheme.	X	
Wildlife:	Loss of wildlife habitat.	Minimal differences between schemes.	The most sensitive wildlife habitat in this reach is upstream of the tunnel re-regulation dam where there is no significant difference between the schemes. The Devil Canyon dam scheme in addition inundates the river valley between the two dam sites resulting in a moderate increase in impacts to wildlife.	Moderate wildlife populations of moose, black bear, weasel, fox, wolverine, other small mammals and songbirds and some riparian cliff habitat for ravens and raptors, in 11 miles of river, would be lost with the dam scheme. Thus, the difference in loss of wildlife habitat is considered moderate and favors the tunnel scheme.	X	
Cultural:	Inundation of archeological sites.	Potential differences between schemes.	Due to the larger area inundated the probability of inundating archeological sites is increased.	Significant archeological sites, if identified, can probably be excavated. Additional costs could range from several hundreds to hundreds of thousands of dollars, but are still considerably less than the additional cost of the tunnel scheme. This concern is not considered a factor in scheme evaluation.	-	-
Land Use:	Inundation of Devil Canyon.	Significant difference between schemes.	The Devil Canyon is considered a unique resource, 80 percent of which would be inundated by the Devil Canyon dam scheme. This would result in a loss of both an aesthetic value plus the potential for white water recreation.	The aesthetic and to some extent the recreational losses associated with the development of the Devil Canyon dam is the main aspect favoring the tunnel scheme. However, current recreational uses of Devil Canyon are low due to limited access. Future possibilites include major recreational development with construction of restaurants, marinas, etc. Under such conditions, neither scheme would be more favorable.	X	

OVERALL EVALUATION: The tunnel scheme has overall a lower impact on the environment.

TABLE B.18 - SOCIAL EVALUATION OF SUSITNA BASIN DEVELOPMENT SCHEMES/PLANS

Social Aspect	Parameter	Tunnel Scheme	Devil Canyon Dam Scheme	High Devil Canyon/ Vee Plan	Watana/Devil Canyon Plan	Remarks
Potential non-renewable resource displacement	Million tons Beluga coal over 50 years	80	110	170	210	Devil Canyon dam scheme potential higher than tunnel scheme. Watana/ Devil Canyon plan higher than High Devil Canyon/ Vee plan.
Impact on state economy	--	All projects would have similar impacts on the state and local economy.				Essentially no difference between plans/schemes.
Impact on local economy	--					
Seismic exposure	Risk of major structural failure	All projects designed to similar levels of safety.				
	Potential impact of failure on human life.	Any dam failures would effect the same downstream population.				
Overall Evaluation	1. Devil Canyon dam superior to tunnel. 2. Watana/Devil Canyon superior to High Devil Canyon/Vee plan.					

TABLE B.19 - ENERGY CONTRIBUTION EVALUATION OF THE DEVIL CANYON DAM AND TUNNEL SCHEMES

Parameter	Dam	Tunnel	Remarks
Total Energy Production Capability			
Annual Average Energy GWH	2850	2240	Devil Canyon dam annually develops 610 GWH and 540 GWH more average and firm energy respectively than the Tunnel scheme.
Firm Annual Energy GWH	2590	2050	
% Basin Potential Developed[1]	43	32	Devil Canyon schemes develops more of the basin potential.
Energy Potential Not Developed GWH	60	380	As currently envisaged, the Devil Canyon dam does not develop 15 ft gross head between the Watana site and the Devil Canyon reservsoir. The tunnel scheme incorporates additional friction losses in tunnels. Also the compensation flow released from re-regulation dam is not used in conjunction with head between re-regulation dam and Devil Canyon.

Notes:

(1) Based on annual average energy. Full potential based on USBR four dam scheme.

TABLE B.20 - OVERALL EVALUATION OF TUNNEL SCHEME AND DEVIL CANYON DAM SCHEME

ATTRIBUTE	SUPERIOR PLAN
Economic	Devil Canyon Dam
Energy Contribution	Devil Canyon Dam
Environmental	Tunnel
Social	Devil Canyon Dam (Marginal)
Overall Evaluation	Devil Canyon dam scheme is superior Tradeoffs made: Economic advantage of dam scheme is judged to outweigh the reduced environmental impact associated with the tunnel scheme.

TABLE B.21 - ENVIRONMENTAL EVALUATION OF WATANA/DEVIL CANYON AND HIGH DEVIL CANYON/VEE DEVELOPMENT PLANS

Environmental Attribute	Plan Comparison	Appraisal Judgement	Plan judged to have the least potential impact	
			HDC/V	W/DC
Ecological: 1) Fisheries	No significant difference in effects on downstream anadromous fisheries. HDC/V would inundate approximately 95 miles of the Susitna River and 28 miles of tributary streams, including the Tyone River. W/DC would inundate approximately 84 miles of the Susitna River and 24 miles of tributary streams, including Watana Creek.	Due to the avoidance of the Tyone River, lesser inundation of resident fisheries habitat and no significant difference in the effects on anadromous fisheries, the W/DC plan is judged to have less impact.		X
2) Wildlife a) Moose	HDC/V would inundate 123 miles of critical winter river bottom habitat. W/DC would inundate 108 miles of this river bottom habitat. HDC/V would inundate a large area upstream of Vee utilized by three sub-populations of moose that range in the northeast section of the basin. W/DC would inundate the Watana Creek area utilized by moose. The condition of this sub-population of moose and the quality of the habitat they are using appears to be decreasing.	Due to the lower potential for direct impact on moose populations within the Susitna, the W/DC plan is judged superior.		X
b) Caribou	The increased length of river flooded, especially upstream from the Vee dam site, would result in the HDC/V plan creating a greater potential division of the Nelchina herd's range. In addition, an increase in range would be directly inundated by the Vee reservoir.	Due to the potential for a greater impact on the Nelchina caribou herd, the HDC/V scheme is considered inferior.		X
c) Furbearers	The area flooded by the Vee reservoir is considered important to some key furbearers, particularly red fox. This area is judged to be more important than the Watana Creek area that would be inundated by the W/DC plan.	Due to the lesser potential for impact on furbearers the W/DC is judged to be superior.		X
d) Birds and Bears	Forest habitat, important for birds and black bears, exist along the valley slopes. The loss of this habitat would be greater with the W/DC plan.	The HDC/V plan is judged superior.	X	
Cultural:	There is a high potential for discovery of archeological sites in the easterly region of the Upper Susitna Basin. The HDC/V plan has a greater potential of affecting these sites. For other reaches of the river the difference between plans is considered minimal.	The W/DC plan is judged to have a lower potential effect on archeological sites.		X

TABLE B.21 (Continued)

Environmental Attribute	Plan Comparison	Appraisal Judgement	Plan judged to have the least potential impact: HDC/V	W/DC
Aesthetic/ Land Use				
	With either scheme, the aesthetic quality of both Devil Canyon and Vee Canyon would be impaired. The HDC/V plan would also inundate Tsusena Falls.	Both plans impact the valley aesthetics. The difference is considered minimal.	-	-
	Due to construction at Vee Dam site and the size of the Vee Reservoir, the HDC/V plan would inherently create access to more wilderness area than would the W/DC plan.	As it is easier to extend access than to limit it, inherent access requirements were considered detrimental and the W/DC plan is judged superior. The ecological sensitivity of the area opened by the HDC/V plan reinforces this judgement.		X

OVERALL EVALUATION: The W/DC plan is judged to be superior to the HDC/V plan. (The lower impact on birds and bears associated with HDC/V plan is considered to be outweighed by all the other impacts which favour the W/DC plan.)

NOTES:

W = Watana Dam
DC = Devil Canyon Dam
HDC = High Devil Canyon Dam
V = Vee Dam

TABLE B.22 - ENERGY CONTRIBUTION EVALUATION OF THE WATANA/DEVIL CANYON AND HIGH DEVIL CANYON/VEE PLANS

Parameter	Watana/ Devil Canyon	High Devil Canyon/Vee	Remarks
Total Energy Production Capability			
Annual Average Energy GWH	6070	4910	Watana/Devil Canyon plan annually develops 1160 GWH and 1650 GWH more average and firm energy repectively than the High Devil Canyon/Vee Plan.
Firm Annual Energy GWH	5520	3870	
% Basin Potential Developed (1)	91	81	Watana/Devil Canyon plan develops more of the basin potential
Energy Potential Not Developed GWH (2)	60	650	As currently conceived, the Watana/-Devil Canyon Plan does not develop 15 ft of gross head between the Watana site and the Devil Canyon reservoir. The High Devil Canyon/Vee Plan does not develop 175 ft gross head between Vee site and High Devil reservoir.

Notes:

(1) Based on annual average energy. Full potential based on USBR four dam schemes.
(2) Includes losses due to unutilized head.

TABLE B.23 - OVERALL EVALUATION OF THE HIGH DEVIL CANYON/VEE AND WATANA/DEVIL CANYON DAM PLANS

ATTRIBUTE	SUPERIOR PLAN
Economic	Watana/Devil Canyon
Energy Contribution	Watana/Devil Canyon
Environmental	Watana/Devil Canyon
Social	Watana/Devil Canyon (Marginal)
Overall Evaluation	Plan with Watana/Devil Canyon is superior Tradeoffs made: None

TABLE B.24: COMBINED WATANA AND DEVIL CANYON OPERATION

Watana Dam Crest Elevation (ft MSL)	Watana* Cost ($ x 10^6)	Devil Canyon* Cost ($ x 10^6)	Total Cost ($ x 10^6)	Average Annual Energy (GWh)
2240 (2215 reservoir elevation)	4,076	1,711	5,787	6,809
2190 (2165 reservoir elevation)	3,785	1,711	5,496	6,586
2140 (2115 reservoir elevation)	3,516	1,711	5,227	6,264

Watana Project alone (prior to year 2002)

Crest Elevation (ft MSL)	Average Annual Energy (GWh)
2240	3,542
2190	3,322
2140	3,071

* Estimated costs in January 1982 dollars, based on preliminary conceptual designs, including relict channel drainage blanket and 20 percent contingencies.

TABLE B.25: PRESENT WORTH OF PRODUCTION COSTS

Watana Dam Crest Elevation (ft MSL)	Present Worth of Production Costs ($ x 10^6
2240 (reservoir elevation 2215)	7,123
2190 (reservoir elevation 2165)	7,052
2140 (reservoir elevation 2115)	7,084

* LTPW in January 1982 dollars.

TABLE B.26: DESIGN DATA AND DESIGN CRITERIA FOR FINAL REVIEW OF LAYOUTS

River Flows

Average flow (over 30 years of record):	7,860 cfs
Probable maximum flood (routed):	326,000 cfs
Maximum inflow with return period of 1:10,000 years:	156,000 cfs
Maximum 1:10,000-year routed discharge:	115,000 cfs
Maximum flood with return period of 1:500 years:	116,000 cfs
Maximum flood with return period of 1:50 years:	87,000 cfs
Reservoir normal maximum operating level:	2215 ft
Reservoir minimum operating level:	2030 ft

Dam

Type:	Rockfill
Crest elevation at point of maximum super elevation:	2240 ft
Height:	890 ft above foundation
Cutoff and foundation treatment:	Core founded on rock; grout curtain and downstream drains
Upstream slope:	2.4H:1V
Downstream slope:	2H:1V
Crest width:	50 ft

Diversion

Cofferdam type:	Rockfill
Cutoff and foundation:	Slurry trench to bedrock
Upstream cofferdam crest elevation:	1585 ft
Downstream cofferdam crest elevation:	1475 ft
Maximum pool level during construction:	1580 ft
Tunnels	Concrete lined,
Final closure:	Mass concrete plugs
Releases during impounding:	6,000 cfs maximum via bypass to outlet structure

Spillway

Design floods:	Passes PMF, preserving integrity of dam with no loss of life Passes routed 1:10,000-year flood with no damage to structures
Main spillway - Capacity:	Routed 1:10,000-year flood with 5 ft surcharge
- Control structure:	Gated ogee crests
Emergency spillway - Capacity:	PMF minus 1:10,000 year flood
- Type:	Fuse plug

Power Intake

Type:	Reinforced concrete
Number of intakes:	6
Draw-off requirements:	Multi-level corresponding to temperature strata
Drawdown:	185 feet

TABLE B.26: (Cont'd)

Penstocks	
Type:	Concrete-lined tunnels with downstream steel liners
Number of penstocks:	6
Powerhouse	
Type:	Underground
Transformer area:	Separate gallery
Control room and administration:	Surface
Access - Vehicle:	Rock tunnel
- Personnel:	Elevator from surface
Power Plant	
Type of turbines:	Francis
Number and rating:	6 x 170 MW
Rated net head:	690 ft
Design flow:	3,500 cfs per unit
Normal maximum gross head:	745 ft
Type of generator:	Vertical synchronous
Rated output:	190 MVA
Power factor:	0.9
Frequency:	60 HZ
Transformers:	13.8-345 kV, 3-phase
Tailrace	
Water passages:	2 concrete-lined tunnels
Surge:	Separate surge chambers
Average tailwater elevation (full generation):	1458 ft

TABLE B.27: EVALUATION CRITIERA

PRELIMINARY REVIEW	INTERMEDIATE REVIEW	FINAL REVIEW
Technical feasibility	Technical feasibility	Technical feasibility
Compatibility of layout with known geological and topographical site features	Compatibility of layout with known geological and topographical site features	Compatibility of layout with known geological and topographical site features
Ease of construction	Ease of construction	Ease of construction
Physical dimensions of component structures in certain locations	--	--
Obvious cost differences of comparable structures	Overall cost	Overall cost
Environmental acceptability	Environmental acceptability	Environmental impact
Operating characteristics	Operating characteristics	Mode of operation of spillways
--	Impact on construction schedule	Impact on construction schedule
--	--	Design and operating limitations for key structures

TABLE B.28: SUMMARY OF COMPARATIVE COST ESTIMATES

INTERMEDIATE REVIEW OF ALTERNATIVE ARRANGEMENTS
(January 1982 \$ x 10^6)

	WP1	WP2	WP3	WP4
Diversion	101.4	112.6	101.4	103.1
Service Spillway	128.2	208.3	122.4	267.2
Emergency Spillway	-	46.9	46.9	-
Tailrace Tunnel	13.1	13.1	13.1	8.0
Credit for Use of Rock in Dam	(11.7)	(31.2)	(18.8)	(72.4)
Total Non-Common Items	231.0	349.7	265.0	305.9
Common Items	1643.0	1643.0	1643.0	1643.0
Subtotal	1874.0	1992.7	1908.0	1948.9
Camp & Support Costs (16%)	299.8	318.8	305.3	311.8
Subtotal	2173.8	2311.5	2213.3	2260.7
Contingency (20%)	434.8	462.3	442.7	452.1
Subtotal	2608.6	1773.8	2656.0	2712.8
Engineering and Administration (12.5%)	326.1	346.7	332.0	339.1
TOTAL	2934.7	3120.5	2988.0	3051.9

TABLE B.29: DESIGN DATA AND DESIGN CRITERIA FOR REVIEW OF ALTERNATIVE LAYOUTS

River Flows	
Average flow (over 30 years of record):	8,960 cfs
Probable maximum flood:	270,000 cfs
Max. flood with return period of 1:10,000 years:	135,000 cfs (after routing through Watana
Maximum flood with return period of 1:500 years:	-
Maximum flood with return period of 1:50 years:	42,000 cfs (after routing through Watana
Reservoir	
Normal maximum operating level:	1455 feet
Reservoir minimum operating level:	1430 feet
Area of reservoir at maximum operating level:	21,000 acres
Reservoir live storage:	180,000 acre feet
Reservoir full storage:	1,100,000 acre feet
Dam	
Type:	Concrete arch
Crest elevation:	1455 feet
Crest length:	-
Maximum height above foundation:	635 feet
Crest width:	20 feet
Diversion	
Cofferdam types:	Rockfill
Upstream cofferdam crest elevation:	960 feet
Downstream cofferdam crest elevation:	900 feet
Maximum pool level during construction:	955 feet
Tunnels:	Concrete lined
Outlet structures:	Low-level structure with slide closure gate
Final closure:	Mass concrete plugs in line with dam grout curtain
Releases during impounding:	2,000 cfs min. via fixed-cone valves
Spillway	
Design floods:	Passes PMF, preserving integrity of dam with no loss of life
	Passes routed 1:10,000-year flood with no damage to structures
Service spillway - capacity:	45,000 cfs
- control structure:	Fixed-cone valves
- energy dissipation:	Five 108-inch diameter fixed-cone valves
Secondary spillway - capacity:	90,000 cfs
- control structure:	Gated, ogee crests
- energy dissipation:	Stilling basin
Emergency spillway - capacity:	pmf minus routed 1:10,000-year flood
- type:	Fuse plug

TABLE B.29: (Cont'd)

Power Intake

Type:	Underground
Transformer area:	Separate gallery
Access	Rock Tunnel
Type of turbines:	Francis
Number and rating:	4 x 140 MW
Rated net head:	550 feet
Maximum gross head:	565 feet approx.
Type of generator:	Vertical synchronous
Rated output:	155 MVA
Power factor:	0.9

TABLE B.30: SUMMARY OF COMPARATIVE COST ESTIMATES

PRELIMINARY REVIEW OF ALTERNATIVE ARRANGEMENTS
(January 1982 $ X 10^6)

Item	DC1	DC2	DC3	DC4
Land Acquisition	22.1	22.1	22.1	22.1
Reservoir	10.5	10.5	10.5	10.5
Main Dam	468.7	468.7	468.7	468.7
Emergency Spillway	25.2	25.2	25.2	25.2
Power Facilities	211.7	211.7	211.7	211.7
Switchyard	7.1	7.1	7.1	7.1
Miscellaneous Structures	9.5	9.5	9.5	9.5
Access Roads & Site Facilities	28.4	28.4	28.4	28.4
Common Items - Subtotal	783.2	783.2	783.2	783.2
Diversion	32.1	32.1	32.1	34.9
Service Spillway	46.8	53.3	50.1	85.2
Saddle Dam	19.9	18.6	18.6	19.9
Non-Common/Items Subtotal	98.8	104.0	100.8	140.0
Total	882.0	887.2	884.0	923.2
Camp & Support Costs (16%)	141.1	141.9	141.4	147.7
Subtotal	1023.1	1029.1	1025.4	1070.9
Contingency (20%)	204.6	205.8	205.1	214.2
Subtotal	1227.7	1234.9	1230.5	1285.1
Engineering & Administration (12.5%)	153.5	154.3	153.8	160.6
Total	1381.2	1389.2	1384.3	1445.7

TABLE B.31: ENERGY POTENTIAL OF WATANA - DEVIL CANYON DEVELOPMENTS FOR DIFFERENT RESERVOIR OPERATING RULES

	ENERGY POTENTIAL GWH											
	WATANA ONLY						WATANA & DEVIL CANYON					
	FIRM ENERGY			AVERAGE ENERGY			FIRM ENERGY			AVERAGE ENERGY		
MONTH	CASE A	C	D	A	C	D	A	C	D	A	C	D
OCT	234	200	172	281	214	178	437	399	334	511	422	346
NOV	270	235	201	348	331	271	502	463	388	543	625	506
DEC	322	276	236	445	397	364	598	547	458	817	751	683
JAN	283	242	208	383	357	325	590	480	403	715	677	618
FEB	228	202	173	318	335	293	452	395	330	599	632	561
MAR	235	201	173	276	330	277	470	398	335	532	629	536
APR	199	165	142	203	214	197	460	332	280	451	419	387
MAY	180	152	131	180	247	174	462	304	286	465	536	399
JUN	170	135	111	175	212	191	492	323	278	478	485	460
JUL	182	209	345	258	267	374	387	471	755	521	579	784
AUG	170	311	531	344	327	545	321	659	1095	598	679	1095
SEP	158	151	155	249	158	166	293	326	390	463	346	395
TOTAL	2632	2479	2578	3459	3389	3354	5394	5099	5332	6793	6781	6768

NOTE: Cases B and C were similar and only Case C was analyzed in detail.

TABLE B.32: AVERAGE ANNUAL AND MONTHLY FLOW AT GAGE IN THE SUSITNA BASIN*

MONTH	STATION (USGS Reference Number)							
	Susitna River at Gold Creek (2920)		Susitna River Near Cantwell (2915)		Susitna River Near Denali (2910)		Maclaren River Near Paxson (2912)	
Drainage Area sq. mi.	6160		4140		950		280	
	%	Mean(cfs)	%	Mean(cfs)	%	Mean(cfs)	%	Mean(cfs)
JANUARY	1	1,453	1	824	1	244	1	96
FEBRUARY	1	1,235	1	722	1	206	1	84
MARCH	1	1,114	1	692	1	188	1	76
APRIL	1	1,367	1	853	1	233	1	87
MAY	12	13,317	10	7,701	6	2,036	7	803
JUNE	24	27,928	26	19,326	22	7,285	25	2,920
JULY	21	23,853	23	16,892	28	9,350	27	3,181
AUGUST	19	21,478	20	14,658	24	8,050	22	2,573
SEPTEMBER	12	13,171	10	7,800	10	3,350	10	1,149
OCTOBER	5	5,639	4	3,033	3	1,122	3	409
NOVEMBER	2	2,467	2	1,449	2	490	1	177
DECEMBER	2	1,773	1	998	1	314	1	118
ANNUAL - cfs	100	9,566	100	6,246	100	2,739	100	973

Period of Record - Gold Creek - 1950-79
Cantwell - 1961-72
Denali - 1957-79
Maclaren - 1957-79

* Ref. USGS Streamflow Data

TABLE B.33: WATANA ESTIMATED NATURAL FLOWS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AVE
1950	4719.9[1]	2083.6	1168.9	815.1	641.7	569.1	680.1	8655.9	16432.1	19193.4	16913.6	7320.4	6599.5
1951	3299.1	1107.3	906.2	808.0	673.0	619.8	1302.2	11649.8	18517.9	19786.6	16478.0	17205.5	7695.1
1952	4592.9	2170.1	1501.0	1274.5	841.0	735.0	803.9	4216.5	25773.4	22110.9	17356.3	11571.0	7745.5
1953	6285.7	2756.8	1281.2	818.9	611.7	670.7	1382.0	15037.2	21469.8	17355.3	16681.6	11513.5	7988.7
1954	4218.9	1599.6	1183.8	1087.8	803.1	638.2	942.6	11696.8	19476.7	16983.6	20420.6	9165.5	7351.4
1955	3859.2	2051.1	1549.5	1388.3	1050.5	886.1	940.8	6718.1	24881.4	23787.9	23537.0	13447.8	8674.8
1956	4102.3	1588.1	1038.6	816.9	754.8	694.4	718.3	12953.3	27171.8	25831.3	19153.4	13194.4	9001.5
1957	4208.0	2276.6	1707.0	1373.0	1189.0	935.0	945.1	10176.2	25275.0	19948.9	17317.7	14841.1	8349.4
1958	6034.9	2935.9	2258.5	1480.6	1041.7	973.5	1265.4	9957.8	22097.8	19752.7	18843.4	5978.7	7718.4
1959	3668.0	1729.5	1115.1	1081.0	949.0	694.0	885.7	10140.6	18329.6	20493.1	23940.4	12466.9	7957.7
1960	5165.5	2213.5	1672.3	1400.4	1138.9	961.1	1069.9	13044.2	13233.4	19506.1	19323.1	16085.6	7901.2
1961	6049.3	2327.8	1973.2	1779.9	1304.8	1331.0	1965.0	13637.9	22784.1	19839.8	19480.2	10146.2	8551.6
1962	4637.6	2263.4	1760.4	1608.9	1257.4	1176.8	1457.4	11333.5	36017.1	23443.7	19887.1	12746.2	9799.1
1963	5560.1	2508.9	1708.9	1308.9	1184.7	883.6	776.6	15299.2	20663.4	28767.4	21011.4	10800.0	9206.1
1964	5187.1	1789.1	1194.7	852.0	781.6	575.2	609.2	3578.8	42841.9	20082.8	14048.2	7524.2	8255.4
1965	4759.4	2368.2	1070.3	863.0	772.7	807.3	1232.4	10966.0	21213.0	23235.9	17394.1	16225.6	8409.0
1966	5221.2	1565.3	1203.6	1060.4	984.7	984.7	1338.4	7094.1	25939.6	16153.5	17390.9	9214.1	7345.9
1967	3269.8	1202.2	1121.6	1102.2	1031.3	889.5	849.7	12555.5	24711.9	21987.3	26104.5	13672.9	9041.5
1968	4019.0	1934.3	1704.2	1617.6	1560.4	1560.4	1576.7	12826.7	25704.0	22082.8	14147.5	7163.6	7991.4
1969	3135.0	1354.9	753.9	619.2	607.5	686.0	1261.6	9313.7	13962.1	14843.5	7771.9	4260.0	4880.8
1970	2403.1	1020.9	709.3	636.2	602.1	624.1	986.4	9536.4	14399.0	18410.1	16263.8	7224.1	6068.0
1971	3768.0	2496.4	1687.4	1097.1	777.4	717.1	813.7	2857.2	27612.8	21126.4	27446.6	12188.9	8549.1
1972	4979.1	2587.0	1957.4	1670.9	1491.4	1366.0	1305.4	15973.1	27429.3	19820.3	17509.5	10955.7	8920.4
1973	4301.2	1977.9	1246.5	1031.5	1000.2	873.9	914.1	7287.0	23859.3	16351.1	18016.7	8099.7	7079.9
1974	3056.5	1354.7	931.6	786.4	689.9	627.3	871.9	12889.0	14780.6	15971.9	13523.7	9786.2	6272.5
1975	3088.8	1474.4	1276.7	1215.8	1110.3	1041.4	1211.2	11672.2	26689.2	23430.4	15126.6	13075.3	8367.7
1976	5679.1	1601.1	876.2	757.8	743.2	690.7	1059.8	8938.8	19994.0	17015.3	18393.5	5711.5	6788.4
1977	2973.5	1926.7	1687.5	1348.7	1202.9	1110.8	1203.4	8569.4	31352.8	19707.3	16807.3	10613.1	8208.6
1978	5793.9	2645.3	1979.7	1577.9	1267.7	1256.7	1408.4	11231.5	17277.2	18305.2	13412.1	7132.6	6947.4
1979	3773.9	1944.9	1312.6	1136.8	1055.4	1101.2	1317.9	12369.3	22904.8	24911.7	16670.7	9096.7	8133.0
1980	6150.0[3]	3525.0[3]	2032.0[3]	1470.0[3]	1233.0[3]	1177.0[3]	1404.0[3]	10140.0[3]	23400.0[2]	26740.0[2]	18000.0[2]	11000.0[2]	8855.9
1981	6458.0[2]	3297.0[2]	1385.0[4]	1147.0[4]	971.0[4]	889.0[4]	1103.0[4]	10406.0[4]	17323.0[2]	27840.0[2]	31435.0[2]	12026.0[2]	9523.3
AVE	4513.1	2052.4	1404.8	1157.3	978.9	898.3	1112.6	10397.6	22922.4	20778.0	18431.4	10670.4	7943.1

Notes: (1) Discharges based on Cantwell and Gold Creek flows unless specified
(2) Watana observed flows
(3) Flows based on Gold Creek
(4) Watana long-term average flows assumed

TABLE B.34: DEVIL CANYON ESTIMATED NATURAL FLOWS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AVE
1950	5758.2	2404.7	1342.5	951.3	735.7	670.0	802.2	10490.7	18468.6	21383.4	18820.6	7950.8	7481.6
1951	3652.0	1231.2	1030.8	905.7	767.5	697.1	1504.6	13218.5	19978.5	21575.9	18530.0	19799.1	8574.2
1952	5221.7	2539.0	1757.5	1483.7	943.2	828.2	878.5	4989.5	30014.2	24861.7	19647.2	13441.1	8883.8
1953	7517.6	3232.6	1550.4	999.6	745.6	766.7	1531.8	17758.3	25230.7	19184.0	19207.0	13928.4	9304.4
1954	5109.3	1921.3	1387.1	1224.2	929.7	729.4	1130.6	15286.0	23188.1	19154.1	24071.6	11579.1	8809.2
1955	4830.4	2506.8	1868.0	1649.1	1275.2	1023.6	1107.4	8390.1	28081.9	26212.8	24959.6	13989.2	9657.8
1956	4647.9	1788.6	1206.6	921.7	893.1	852.3	867.3	15979.0	31137.1	29212.0	22609.8	16495.8	10550.9
1957	5235.3	2773.8	1986.6	1583.2	1388.9	1105.4	1109.0	12473.6	28415.4	22109.6	19389.2	18029.0	9633.3
1958	7434.5	3590.4	2904.9	1792.0	1212.2	1085.7	1437.4	11849.2	24413.5	21763.1	21219.8	6988.8	8807.6
1959	4402.8	1999.8	1370.9	1316.9	1179.1	877.9	1119.9	13900.9	21537.7	23390.4	28594.4	15329.6	9585.0
1960	6060.7	2622.7	2011.5	1686.2	1340.2	1112.8	1217.8	14802.9	14709.8	21739.3	22066.1	18929.9	9025.0
1961	7170.9	2759.9	2436.6	2212.0	1593.6	1638.9	2405.4	16030.7	27069.3	22880.6	21164.4	12218.6	9965.1
1962	5459.4	2544.1	1978.7	1796.0	1413.4	1320.3	1613.4	12141.2	40679.7	24990.6	22241.8	14767.2	10912.2
1963	6307.7	2696.0	1896.0	1496.0	1387.4	958.4	810.9	17697.6	24094.1	32388.4	22720.5	11777.2	10352.5
1964	5998.3	2085.4	1387.1	978.0	900.2	663.8	696.5	4046.9	47816.4	21926.0	15585.8	8840.0	9243.7
1965	5744.0	2645.1	1160.8	925.3	828.8	866.9	1314.4	12267.1	24110.3	26195.7	19789.3	18234.2	9506.8
1966	6496.5	1907.8	1478.4	1278.7	1187.4	1187.4	1619.1	8734.0	30446.3	18536.2	20244.6	10844.3	8663.4
1967	3844.0	1457.9	1364.9	1357.9	1268.3	1089.1	1053.7	14435.5	27796.4	25081.2	30293.0	15728.2	10397.5
1968	4585.3	2203.5	1929.7	1851.2	1778.7	1778.7	1791.0	14982.4	29462.1	24871.0	16090.5	8225.9	9129.2
1969	3576.7	1531.8	836.3	686.6	681.8	769.6	1421.3	10429.9	14950.7	15651.2	8483.6	4795.5	5317.9
1970	2866.5	1145.7	810.0	756.9	708.7	721.8	1046.6	10721.6	17118.9	21142.2	18652.8	8443.5	7011.3
1971	4745.2	3081.8	2074.8	1318.8	943.6	866.8	986.2	3427.9	31031.0	22941.6	30315.9	13636.0	9614.1
1972	5537.0	2912.3	2312.6	2036.1	1836.4	1659.8	1565.5	19776.8	31929.8	21716.5	18654.1	11884.2	10151.8
1973	4638.6	2154.8	1387.0	1139.8	1128.6	955.0	986.7	7896.4	26392.6	17571.8	19478.1	8726.0	7704.6
1974	3491.4	1462.9	997.4	842.7	745.9	689.5	949.1	15004.6	16766.7	17790.0	15257.0	11370.1	7113.9
1975	3506.8	1619.4	1486.5	1408.8	1342.2	1271.9	1456.7	14036.5	30302.6	26188.0	17031.6	15154.7	9567.1
1976	7003.3	1853.0	1007.9	896.8	876.2	825.2	1261.2	11305.3	22813.6	18252.6	19297.7	6463.3	7654.7
1977	3552.4	2391.7	2147.5	1657.4	1469.7	1361.0	1509.8	11211.9	35606.7	21740.5	18371.2	11916.1	9411.3
1978	6936.3	3210.8	2371.4	1867.9	1525.0	1480.6	1597.1	11693.4	18416.8	20079.0	15326.5	8080.4	7715.4
1979	4502.3	2324.3	1549.4	1304.1	1203.6	1164.7	1402.8	13334.0	24052.4	27462.8	19106.7	10172.4	8965.0
1980*	6900.0	3955.0	2279.0	1649.0	1383.0	1321.0	1575.0	11377.0	26255.0	30002.0	20196.0	12342.0	9936.2
1981*	7246.0	3699.0	1554.0	1287.0	1089.0	997.0	1238.0	11676.0	19436.0	31236.0	35270.0	13493.0	10685.1
AVE	5311.8	2382.9	1652.0	1351.9	1146.9	1041.8	1281.5	12230.2	25991.3	23100.9	20709.0	12299.2	9041.6

* Discharges based on Watana flows

TABLE B.35: PEAK FLOWS OF RECORD

Gold Creek		Cantwell		Denali		Maclaren	
Date	Peak ft^3/s	Date	Peak ft^3/s	Date	Peak ft^3/s	Date	Peak ft^3/s
8/25/59	62,300	6/23/61	30,500	8/18/63	17,000	9/13/60	8,900
6/15/62	80,600	6/15/62	47,000	6/07/64	16,000	6/14/62	6,650
6/07/64	90,700	6/07/64	50,500	9/09/65	15,800	7/18/65	7,350
6/06/66	63,600	8/11/70	20,500	8/14/67	28,200	8/14/67	7,600
8/15/67	80,200	8/10/71	60,000	7/27/68	19,000	8/10/71	9,300
8/10/71	87,400	6/22/72	45,000	8/08/71	38,200	6/17/72	7,100

TABLE B.36: ESTIMATED FLOOD PEAKS IN SUSITNA RIVER

Location	Peak Inflow in Cfs for Recurrence Interval in Years				
	1:2	1:50	1:100	1:10,000	PMF
Gold Creek	48,000	105,000	118,000	200,000	408,000
Watana Damsite	42,000	82,000	92,000	156,000	326,000
Devil Canyon Damsite (Routed Peak Inflow with Watana)	12,600	43,000	61,000	165,000	346,000

TABLE B.37: ESTIMATED EVAPORATION LOSSES - WATANA AND DEVIL CANYON RESERVOIRS

Month	WATANA		DEVIL CANYON		Average Monthly Air Temperature (°C)		
	Pan Evaporation (inches)	Reservoir Evaporation (inches)	Pan Evaporation (inches)	Reservoir Evaporation (inches)	Watana[1]	Devil Canyon[2]	Talkeetna[3]
January	0.0	0.0	0.0	0.0	- 2.5	- 4.5	-13.0
February	0.0	0.0	0.0	0.0	- 7.3	- 5.0	- 9.3
March	0.0	0.0	0.0	0.0	- 1.8	- 4.3	- 6.7
April	0.0	0.0	0.0	0.0	- 1.8	- 2.5	0.7
May	3.6	2.5	3.9	2.7	8.7	6.1	7.0
June	3.4	2.4	3.8	2.7	10.0	9.2	12.6
July	3.3	2.3	3.7	2.6	13.7	11.9	14.4
August	2.5	1.8	2.7	1.9	12.5	N/A	12.7
September	1.5	1.0	1.7	1.2	N/A	4.8	7.8
October	0.0	0.0	0.0	0.0	0.2	- 1.8	0.2
November	0.0	0.0	0.0	0.0	- 5.1	- 7.2	- 7.8
December	0.0	0.0	0.0	0.0	-17.9	-21.1	-12.7
Annual Evap.	14.3	10.0	15.8	11.1			

1 Based on data - April 1980-June 1981
2 Based on data - July 1980-June 1981
3 Based on data - January 1941-December 1980

TABLE B.38: MONTHLY FLOW REQUIREMENT AT GOLD CREEK

Month	Monthly Flow (cfs) Case A	B and C	D
Oct	1000	5500	5500
Nov	900	1200	1200
Dec	900	1200	1200
Jan	900	1200	1200
Feb	900	1200	1200
Mar	900	1200	1200
Apr	900	1200	1200
May	1000	6000	6000
Jun	2000	7000	7000
Jul	2000	9500	7000/1900 (1)
Aug	2000	12000	19000
Sep	1000	12000	12000

(1) Split Month: 7000 cfs to 15th then 19000 cfs to month end.

TABLE B.39: REQUIRED FLOW RELEASES AT WATANA

TABLE B.40: REQUIRED FLOW RELEASES AT DEVIL CANYON

To be included after selection of operation schedule and scheme

TABLE B.41: WATER APPROPRIATIONS WITHIN ONE MILE OF THE SUSITNA RIVER

LOCATION*	ADDITIONAL NUMBER	TYPE	SOURCE (DEPTH)	AMOUNT	DAYS OF USE
CERTIFICATE					
T19N R5W	45156	Single-family dwelling	well (?)	650 gpd	365
		general crops	same source	0.5 ac-ft/yr	91
T25N R5W	43981	Single-family dwelling	well (90 ft)	500 gpd	365
T26N R5W	78895	Single-family dwelling	well (20 ft)	500 gpd	365
	200540	Grade school	well (27 ft)	910 gpd	334
	209233	Fire station	well (34 ft)	500 gpd	365
T27N R5W	200180	Single-family dwelling	unnamed stream	200 gpd	365
		Lawn & garden irrigation	same source	100 gpd	153
	200515	Single-family dwelling	unnamed lake	500 gpd	365
	206633	Single-family dwelling	unnamed lake	75 gpd	365
	206930	Single-family dwelling	unnamed lake	250 gpd	365
	206931	Single-family dwelling	unnamed lake	250 gpd	365
PERMIT					
	206929	General crops	unnamed creek	1 ac-ft/yr	153
T30N R3W	206735	Single-family dwelling	unnamed stream	250 gpd	365
PENDING					
	209866	Single-family dwelling	Sherman Creek	75 gpd	365
		Lawn & garden irrigation	same source	50 gpd	183

*All locations are within the Seward Meridian.

TABLE B.42: TURBINE OPERATING CONDITIONS

	Watana	Devil Canyon
Maximum net head	728 feet	597 feet
Minimum net head	576 feet	238 feet
Design head	680 feet	575 feet
Rated head	680 feet	575 feet
Turbine flow at rated head, cfs	3550 cfs	3800 cfs
Turbine efficiency at design head	91%	91%
Turbine-generating rating at rated head	181,500 kW	164,000 kW

TABLE B.43: HISTORICAL ANNUAL GROWTH RATES OF ELECTRIC UTILITY SALES

Period	U.S.	Anchorage and Fairbanks Areas
1940 - 1950	8.8%	20.5%
1950 - 1960	8.7%	15.3%
1960 - 1970	7.3%	12.9%
1970 - 1978	4.6%	11.7%
1970 - 1973	6.7%	13.1%
1973 - 1978	3.5%	10.9%
1940 - 1978	7.3%	15.2%

TABLE B.44: ANNUAL GROWTH RATES IN UTILITY CUSTOMERS AND CONSUMPTION PER CUSTOMER

	Greater Anchorage		Greater Fairbanks		U.S.	
	Customers (Thousands)	Consumption per Customer (MWh)	Customers (Thousands)	Consumption per Customer (MWh)	Customers (Millions)	Consumption per Customer (MWh)
Residential						
1965	27	6.4	8.2	4.8	57.6	4.9
1978	77	10.9	17.5	10.2	77.8	8.8
Annual Growth Rate (%)	8.4	4.2	6.0	6.0	2.3	4.6
Commercial						
1965	4.0	-	1.3	-	7.4	-
1978	10.2	-	2.9	-	9.1	
Annual Growth Rate (%)	7.5	-	6.4	-	1.6	-

TABLE B.45: UTILITY SALES BY RAILBELT REGIONS

Year	Greater Anchorage Sales GWh	Greater Anchorage Sales Regional Share	Greater Anchorage No. of Customers[1] (Thousands)	Greater Fairbanks Sales GWh	Greater Fairbanks Sales Regional Share	Greater Fairbanks No. of Customers[1] (Thousands)	Glennallen-Valdez Sales GWh	Glennallen-Valdez Sales Regional Share	Glennallen-Valdez No. of Customers[1] (Thousands)	Railbelt Total Sales GWh	Railbelt Total No. of Customers[1] (Thousands)
1965	369	78%	31.0	98	21%	9.5	6	1%	.6	473	41.1
1966	415		32.2	108		9.6	NA		NA	523	41.8
1967	461		34.4	66		NA	NA		NA	527	34.4
1968	519		39.2	141		10.8	NA		NA	661	30.0
1969	587		42.8	170		11.6	NA		NA	758	54.4
1970	684	75%	46.9	213	24%	12.6	9	1%	.8	907	60.3
1971	797		49.5	251		13.1	10		.9	1059	63.5
1972	906		54.1	262		13.5	6		.4	1174	68.0
1973	1010		56.1	290		13.9	11		1.0	1311	71.0
1974	1086		61.8	322		15.5	14		1.3	1422	78.6
1975	1270	75%	66.1	413	24%	16.2	24	1%	1.9	1707	84.2
1976	1463		71.2	423		17.9	33		2.2	1920	91.3
1977	1603		81.1	447		20.0	42		2.1	2092	103.2
1978	1747	79%	87.2	432	19%	20.4	38	2%	2.0	2217	109.6
Annual Growth	12.7%		8.2%	12.1%		6.1%	13.9%		9.7%	12.6%	7.8%

NOTES:

(1) Includes residential and commercial users only, but not miscellaneous users.
Source: Federal Energy Regulatory Commission, Power System Statement.
NA: Not Available.

TABLE B.46: SUMMARY OF ISER RAILBELT ELECTRICITY PROJECTIONS

	Utility Sales to All Consuming Sectors (GWh)						Military Net Generation (GWh)	Self-Supplied Industry Net Generation (GWh)			
Year	LES-GL[1] Bound	LES-GM	MES-GM (Base Case)	MES-GM with Price Induced Shift	HES-GM	HES-GH[1] Bound	MES-GM (Base Case)	LES-GM	MES-GM (Base Case)	MES-GM with Price Induced Shift	HES-GM
1980	2390	2390	2390	2390	2390	2390	334	414	414	414	414
1985	2798	2921	3171	3171	3561	3707	334	414	571	571	847
1990	3041	3236	3599	3599	4282	4443	334	414	571	571	981
1995	3640	3976	4601	4617	5789	6317	334	414	571	571	981
2000	4468	5101	5730	6525	7192	8010	334	414	571	571	981
2005	4912	5617	6742	8219	9177	10596	334	414	571	571	981
2010	5442	6179	7952	10142	11736	14009	334	414	571	571	981
Average Annual Growth Rate (%)											
1980-1990	2.44	3.08	4.18	4.18	6.00	6.40	0.0	0.0	3.27	3.27	9.0
1990-2000	3.92	4.66	4.76	6.13	5.32	6.07	0.0	0.0	0.0	0.0	0.0
2000-2010	1.99	1.94	3.33	4.51	5.02	5.75	0.0	0.0	0.0	0.0	0.0
1980-2010	2.78	3.22	4.09	4.94	5.45	6.07	0.0	0.0	1.08	1.08	2.92

NOTES:

Lower Bound = Estimates for LES-GL
Upper Bound = Estimates for HES-GH

LES = Low Economic Growth
MES = Medium Economic Growth
HES = High Economic Growth
GL = Low Government Expenditure
GM = Moderate Government Expenditure
GH = High Government Expenditure

(1) Results generated by Acres, all others by ISER.

TABLE B.47: FORECAST TOTAL GENERATION AND PEAK LOADS - TOTAL RAILBELT REGION[1]

	ISER Low (LES-GM)[2]		ISER Medium (MES-GM)		ISER High (HES-GM)	
Year	Generation (GWh)	Peak Load (MW)	Generation (GWh)	Peak Load (MW)	Generation (GWh)	Peak Load (MW)
1978	3323	606	3323	606	3323	606
1980	3522	643	3522	643	4135	753
1985	4141	757	4429	808	5528	995
1990	4503	824	4922	898	6336	1146
1995	5331	977	6050	1105	8013	1456
2000	6599	1210	7327	1341	9598	1750
2005	7188	1319	8471	1551	11843	2158
2010	7822	1435	9838	1800	14730	2683
Percent Growth/Yr. 1978-2010	2.71	2.73	3.45	3.46	4.76	4.76

NOTES:

(1) Includes net generation from military and self-supplied industry sources.

(2) All forecasts assume moderate government expenditure.

TABLE B.48: ISER 1980 RAILBELT REGION LOAD AND ENERGY FORECASTS USED FOR GENERATION PLANNING STUDIES FOR DEVELOPMENT SELECTION[5]

	LOAD CASE											
	Low Plus Load Management and Conservation (LES-GL Adjusted)[1]			Low (LES-GL)[2]			Medium (MES-GM)[3]			High (HES-GH)[4]		
Year	MW	GWh	Load Factor	MW	GWh	Load Factor	MW	GWh	Load Factor	MW	GWh	Load Factor
1980	510	2790	62.5	510	2790	62.4	510	2790	62.4	510	2790	62.4
1985	560	3090	62.8	580	3160	62.4	650	3570	62.6	695	3860	63.4
1990	620	3430	63.2	640	3505	62.4	735	4030	62.6	920	5090	63.1
1995	685	3810	63.5	795	4350	62.3	945	5170	62.5	1295	7120	62.8
2000	755	4240	63.8	950	5210	62.3	1175	6430	62.4	1670	9170	62.6
2005	835	4690	64.1	1045	5700	62.2	1380	7530	62.3	2285	12540	62.6
2010	920	5200	64.4	1140	6220	62.2	1635	8940	62.4	2900	15930	62.7

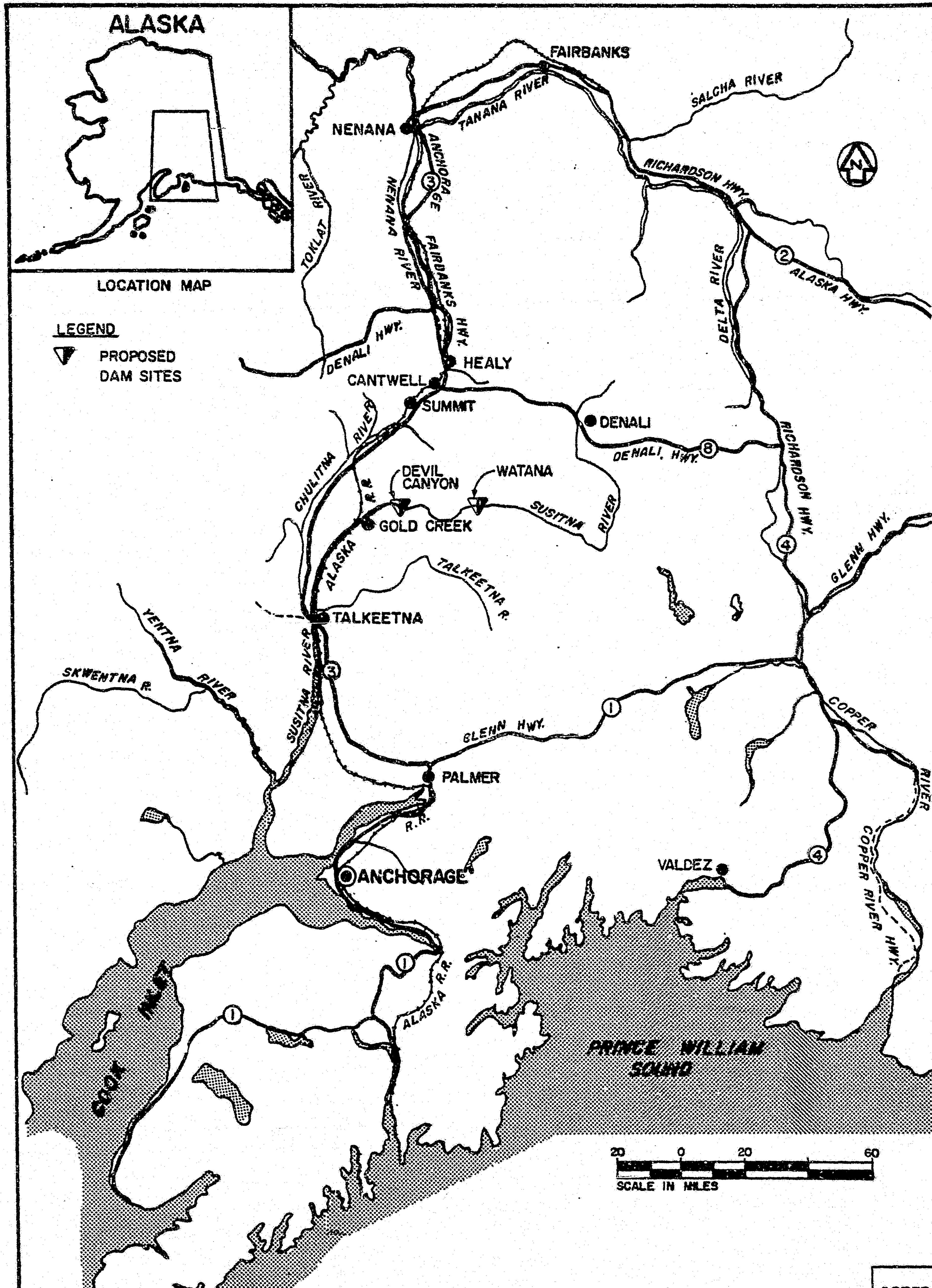
Notes:

(1) LES-GL: Low economic growth/low government expenditure with load management and conservation.
(2) LES-GL: Low economic growth/low government expenditure.
(3) MES-GM: Medium economic growth/moderate government expenditure.
(4) HES-GH: High economic growth/high government expenditure.
(5) Excludes reserve requirements. Energy figures are for net generation.

TABLE B.49: DECEMBER 1981 BATTELLE PNL RAILBELT REGION LOAD AND ENERGY FORECASTS USED FOR GENERATION PLANNING STUDIES

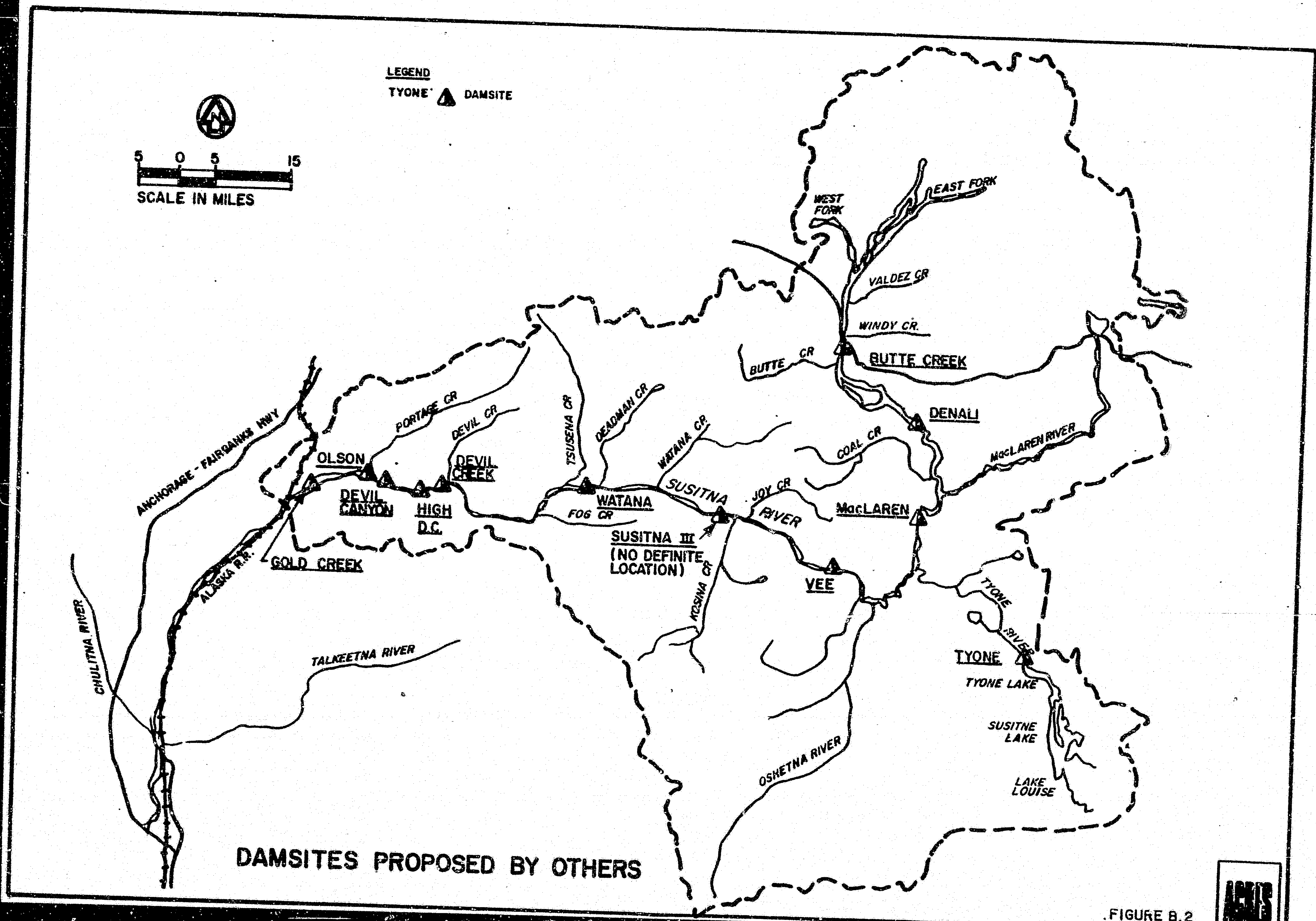
	LOAD CASE								
	Medium			Low			High		
Year	MW	GWh	Load Factor	MW	GWh	Load Factor	MW	GWh	Load Factor
1981	574	2893	57.5	568	2853	57.3	598	3053	58.3
1985	687	3431	57.8	642	3234	57.5	794	4231	60.8
1990	892	4456	57.0	802	3999	56.9	1098	5703	59.3
1995	983	4922	57.1	849	4240	57.0	1248	6464	59.1
2000	1084	5469	57.4	921	4641	57.4	1439	7457	59.0
2005	1270	6428	57.8	1066	5358	57.4	1769	9148	59.0
2010	1537	7791	57.9	1245	6303	57.8	2165	11,435	60.3
Average Annual Growth Rate(%)									
1981-1990	5.0	4.9		3.9	3.8		7.0	7.2	
1990-2000	2.0	2.1		1.4	1.5		2.7	2.7	
2001-2010	3.6	3.6		3.1	3.1		4.2	4.4	
1981-2010	3.5	3.5		2.7	2.8		4.5	4.6	

Note: Excludes reserve requirements. Energy figures are for net generation.



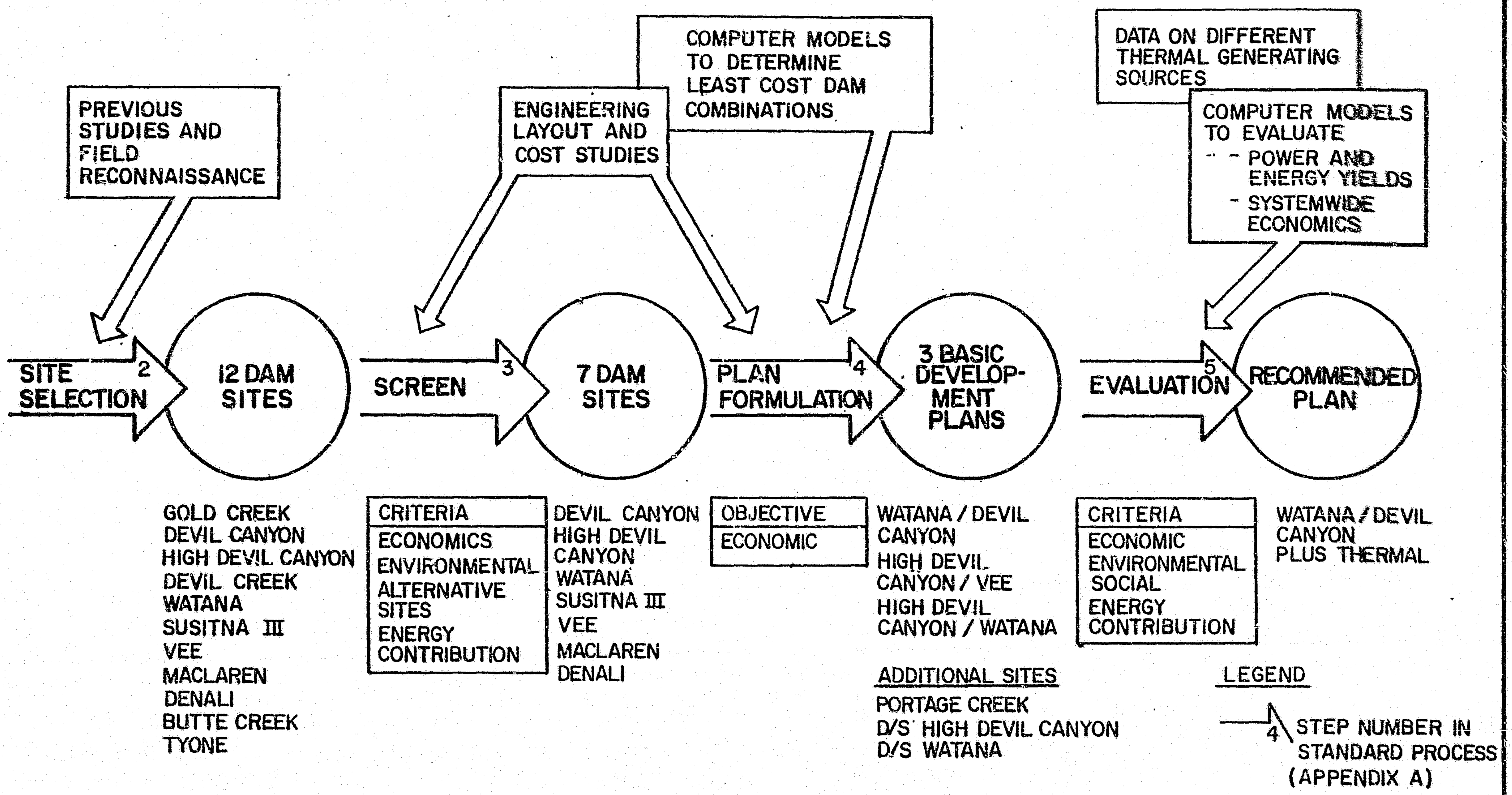
LOCATION MAP

FIGURE B.1



DAMSITES PROPOSED BY OTHERS

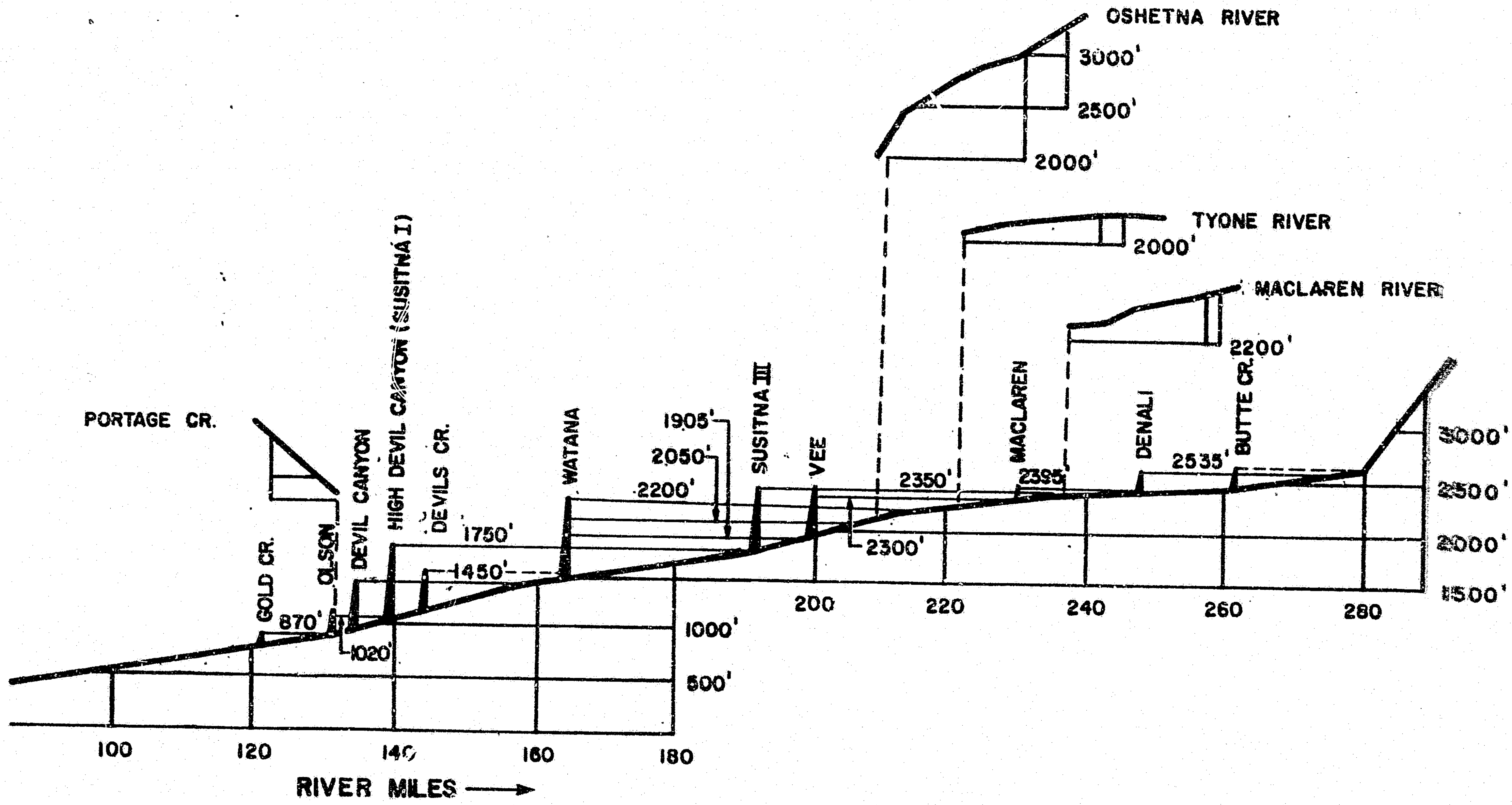
FIGURE B.2



SUSITNA BASIN PLAN FORMULATION AND SELECTION PROCESS

ACRES

FIGURE B.3



PROFILE THROUGH ALTERNATIVE SITES

FIGURE B.4

	GOLD CREEK	OLSON	DEVIL CANYON	HIGH DEVIL CANYON	DEVIL CREEK	WATANA	SUSITNA III	VEE	MACLAREN	DENALI	BUTTE CREEK	TYONE
GOLD CREEK												
OLSON			890 (30)									
DEVIL CANYON				1030 (160)	1070 (200)							
HIGH DEVIL CANYON						1465 (445)						
DEVIL CREEK						1465 (405)						
WATANA							1810 (355)	1925 (470)				
SUSITNA III								1925 (125)	2320 (520)			
VEE									2320 (405)			
MACLAREN												
DENALI												
BUTTE CREEK												
TYONE												

LEGEND

COMPATIBLE ALTERNATIVES

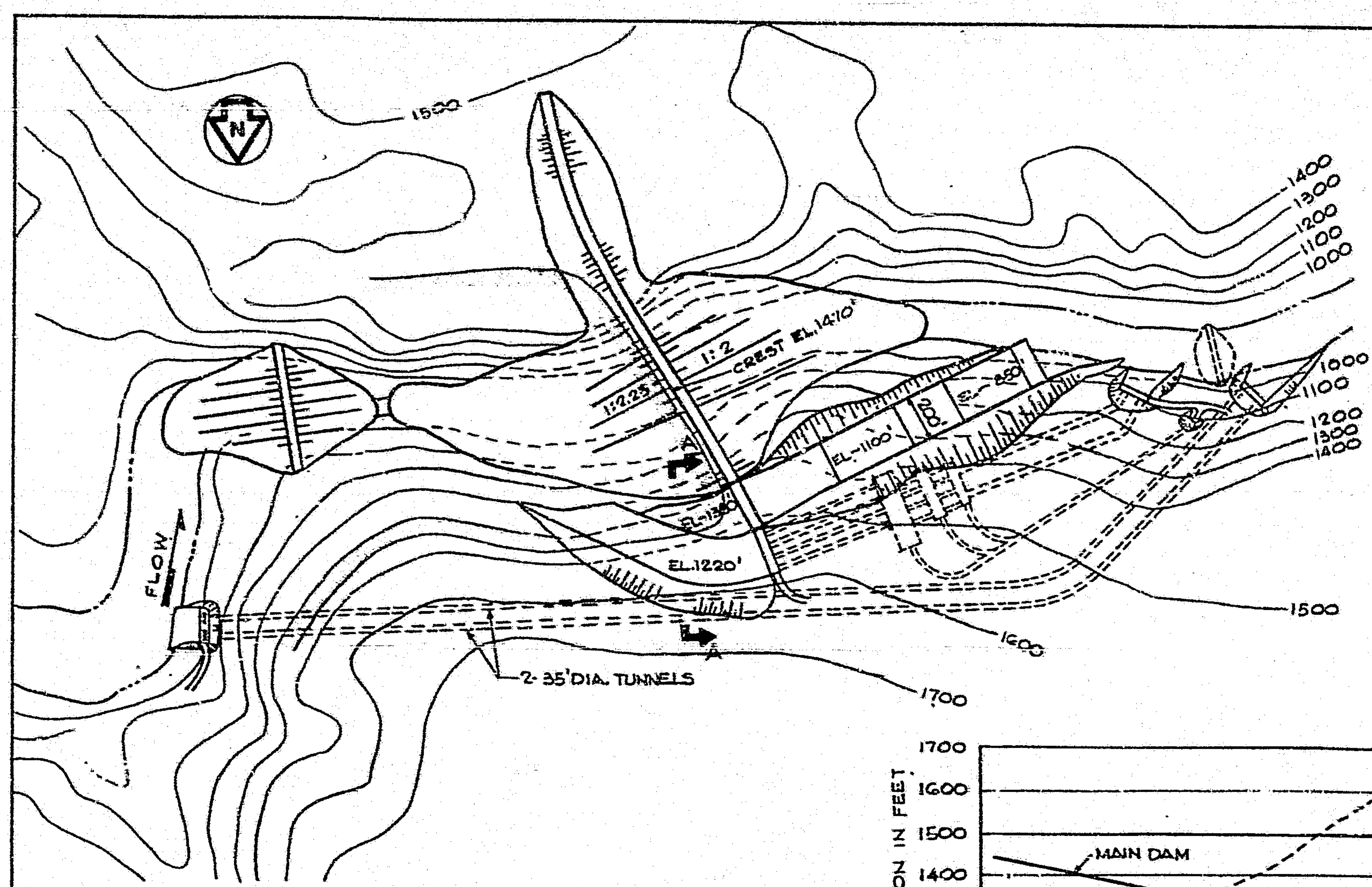
MUTUALLY EXCLUSIVE ALTERNATIVES

1465 (405) — DAM IN COLUMN IS MUTUALLY EXCLUSIVE IF FULL SUPPLY LEVEL OF DAM IN ROW EXCEEDS THIS VALUE - FT.

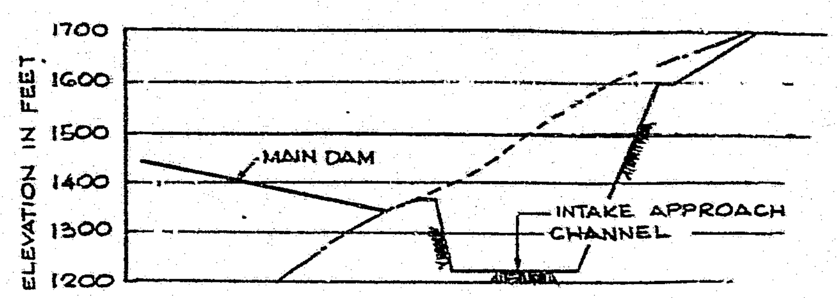
VALUE IN BRACKET REFERS TO APPROXIMATE DAM HEIGHT.

MUTUALLY EXCLUSIVE DEVELOPMENT ALTERNATIVES

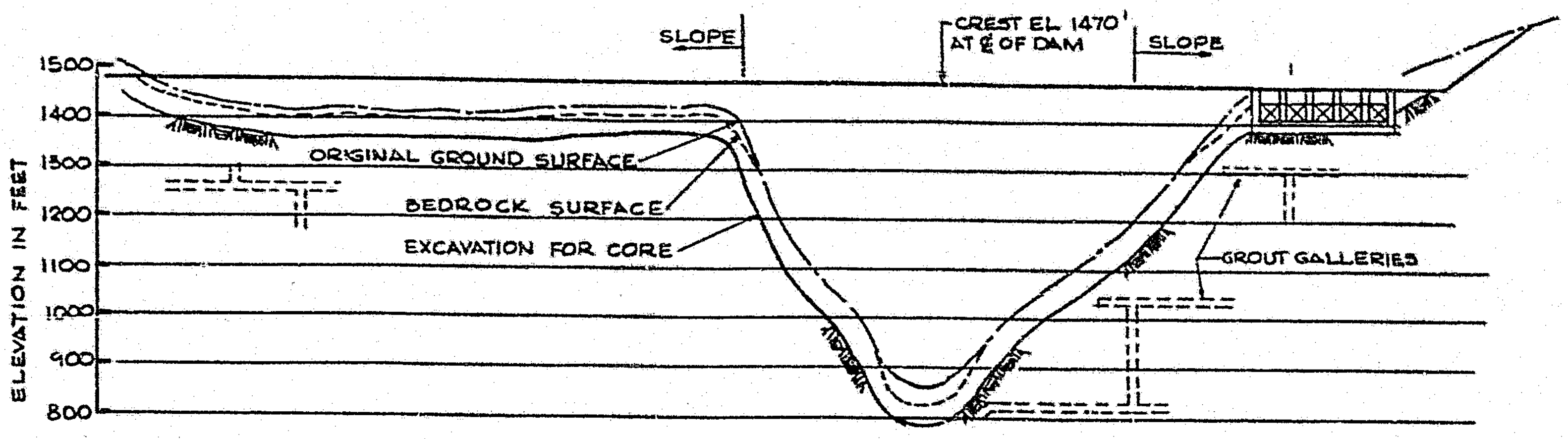
FIGURE B.5



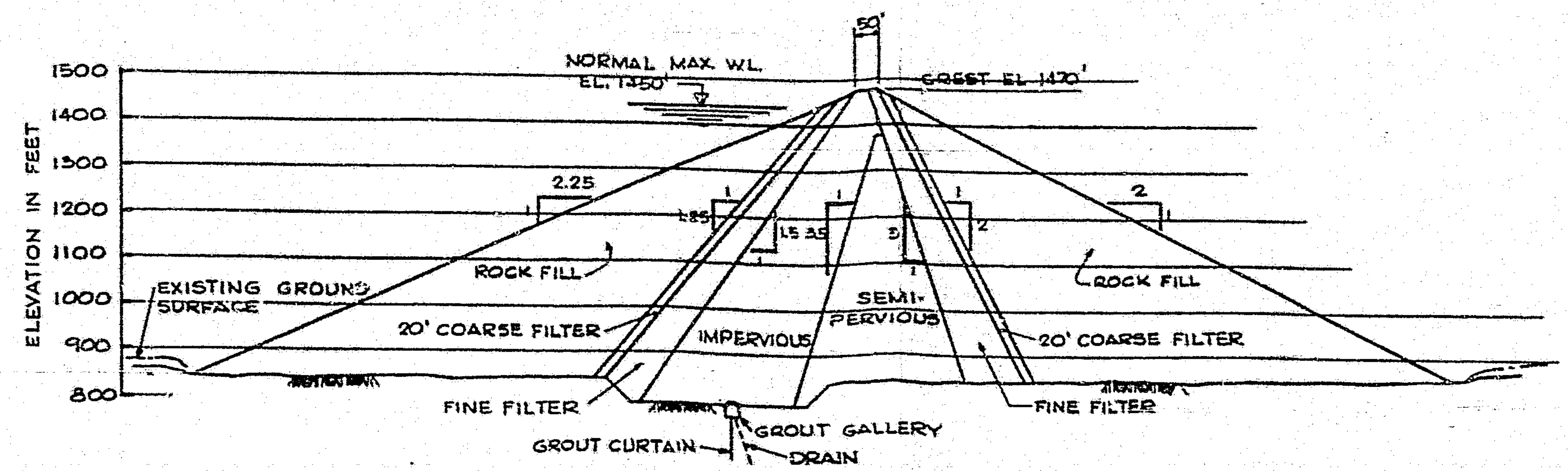
GENERAL ARRANGEMENT
SCALE: A



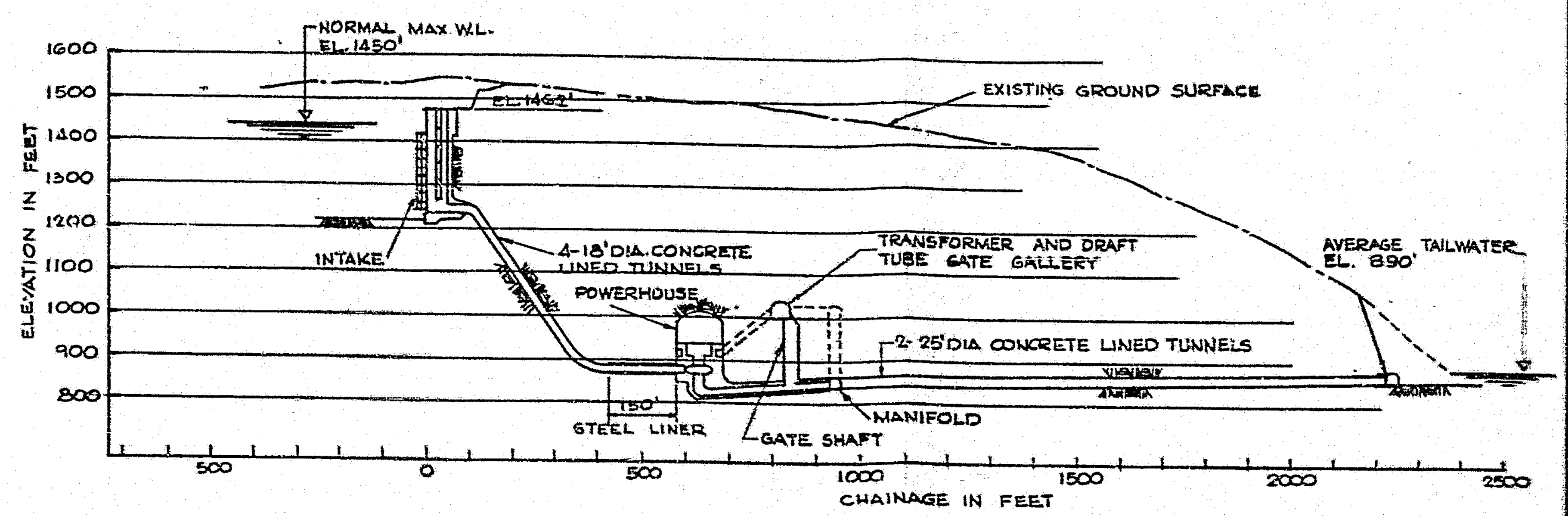
SECTION A-A
SCALE: B



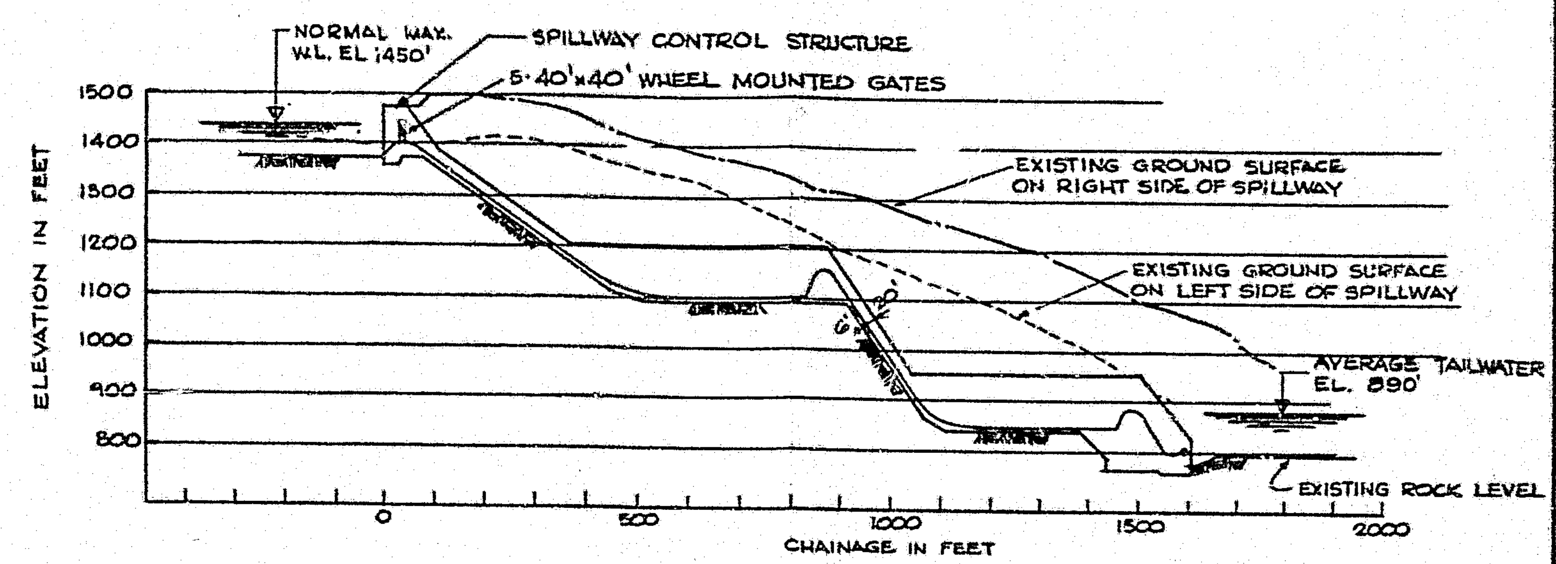
LONGITUDINAL SECTION THRU ¢ OF DAM
SCALE: B



SECTION THRU DAM
SCALE: B



POWER FACILITIES PROFILE
SCALE: B



SPILLWAY PROFILE
SCALE: B

SCALE A 0 400 800 FEET

SCALE B 0 200 400 FEET

FIGURE B.6

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DEVIL CANYON
HYDRO DEVELOPMENT
FILL DAM

DATE DEC. 1981 | SCALE AS SHOWN | SK-5700-CG-102

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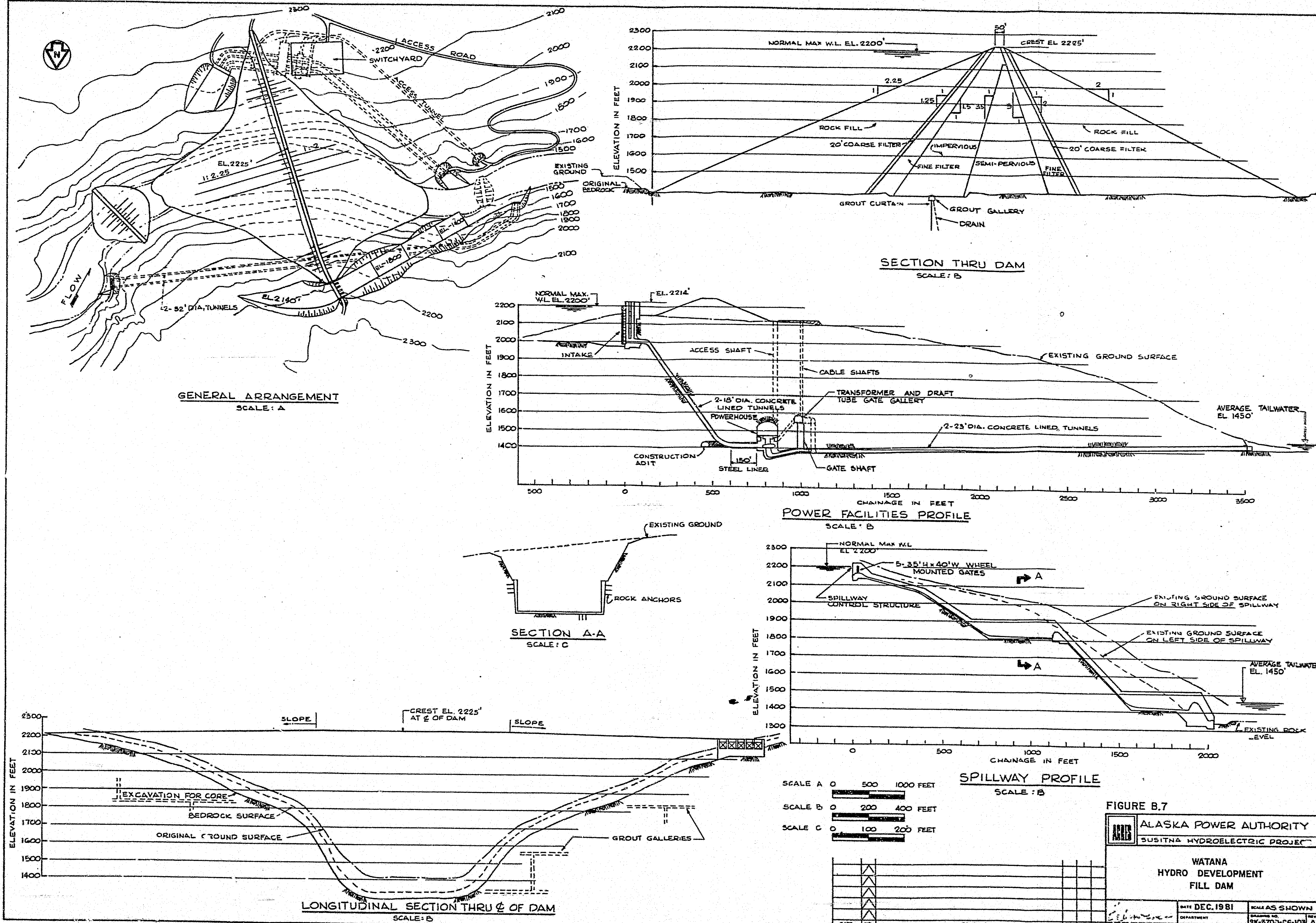


GENERAL ARRANGEMENT
SCALE: A
ACCESS ROAD
SWITCHYARD
ACCESS TUNNEL
EL. 2225'
1:2.25
EXISTING GROUND
ORIGINAL BEDROCK
FLOW
2-32' DIA. TUNNELS
EL. 2140'
EL. 1800
SECTION THRU DAM
SCALE: B
NORMAL MAX W.L. EL. 2200'
CREST EL 2225'
ROCK FILL
20' COARSE FILTER
IMPERVIOUS
FINE FILTER
SEMI-PERVIOUS
FINE FILTER
20' COARSE FILTER
ROCK FILL
GROUT CURTAIN
GROUT GALLERY
DRAIN
ELEVATION IN FEET
POWER FACILITIES PROFILE
SCALE: B
NORMAL MAX. W.L. EL. 2200'
EL. 2214'
INTAKE
ACCESS SHAFT
CABLE SHAFTS
EXISTING GROUND SURFACE
2-18' DIA. CONCRETE LINED TUNNELS
TRANSFORMER AND DRAFT TUBE GATE GALLERY
POWERHOUSE
2-23' DIA. CONCRETE LINED TUNNELS
AVERAGE TAILWATER EL 1450'
CONSTRUCTION ADIT
150'
STEEL LINER
GATE SHAFT
CHAINAGE IN FEET
SECTION A-A
SCALE: C
EXISTING GROUND
ROCK ANCHORS
SPILLWAY PROFILE
SCALE: B
NORMAL MAX W.L. EL 2200'
5-35'H x 40'W WHEEL MOUNTED GATES
SPILLWAY CONTROL STRUCTURE
EXISTING GROUND SURFACE ON RIGHT SIDE OF SPILLWAY
EXISTING GROUND SURFACE ON LEFT SIDE OF SPILLWAY
AVERAGE TAILWATER EL. 1450'
EXISTING ROCK LEVEL
CHAINAGE IN FEET
LONGITUDINAL SECTION THRU ℄ OF DAM
SCALE: B
CREST EL. 2225' AT ℄ OF DAM
SLOPE
EXCAVATION FOR CORE
BEDROCK SURFACE
ORIGINAL GROUND SURFACE
GROUT GALLERIES
SCALE A 0 500 1000 FEET
SCALE B 0 200 400 FEET
SCALE C 0 100 200 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA
HYDRO DEVELOPMENT
FILL DAM
DATE DEC. 1981
SCALE AS SHOWN
SK-5700-C6-K3
DATE NO. REVISIONS CH. APP. APP.
ACRES AMERICAN INCORPORATED

FIGURE B.7

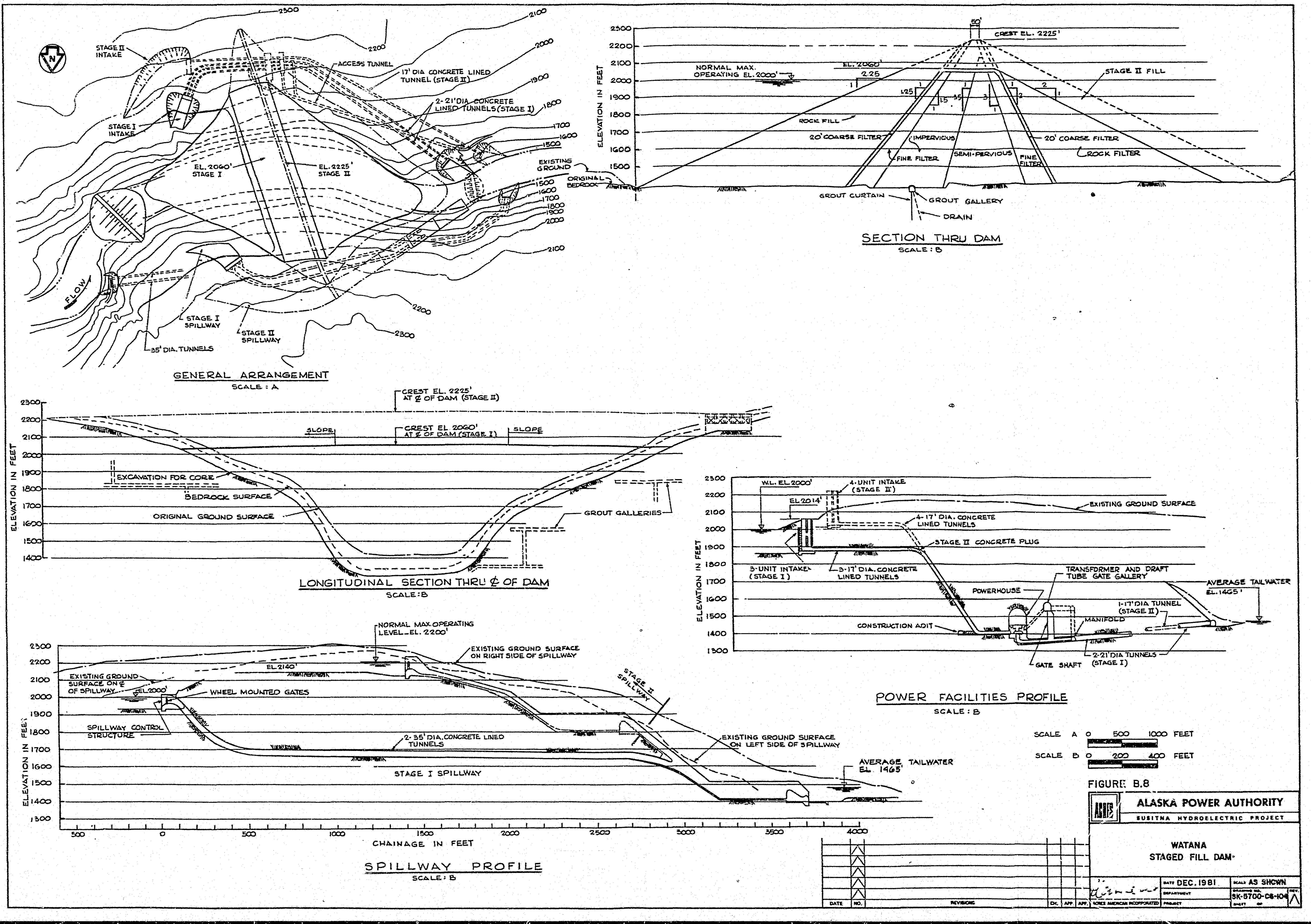
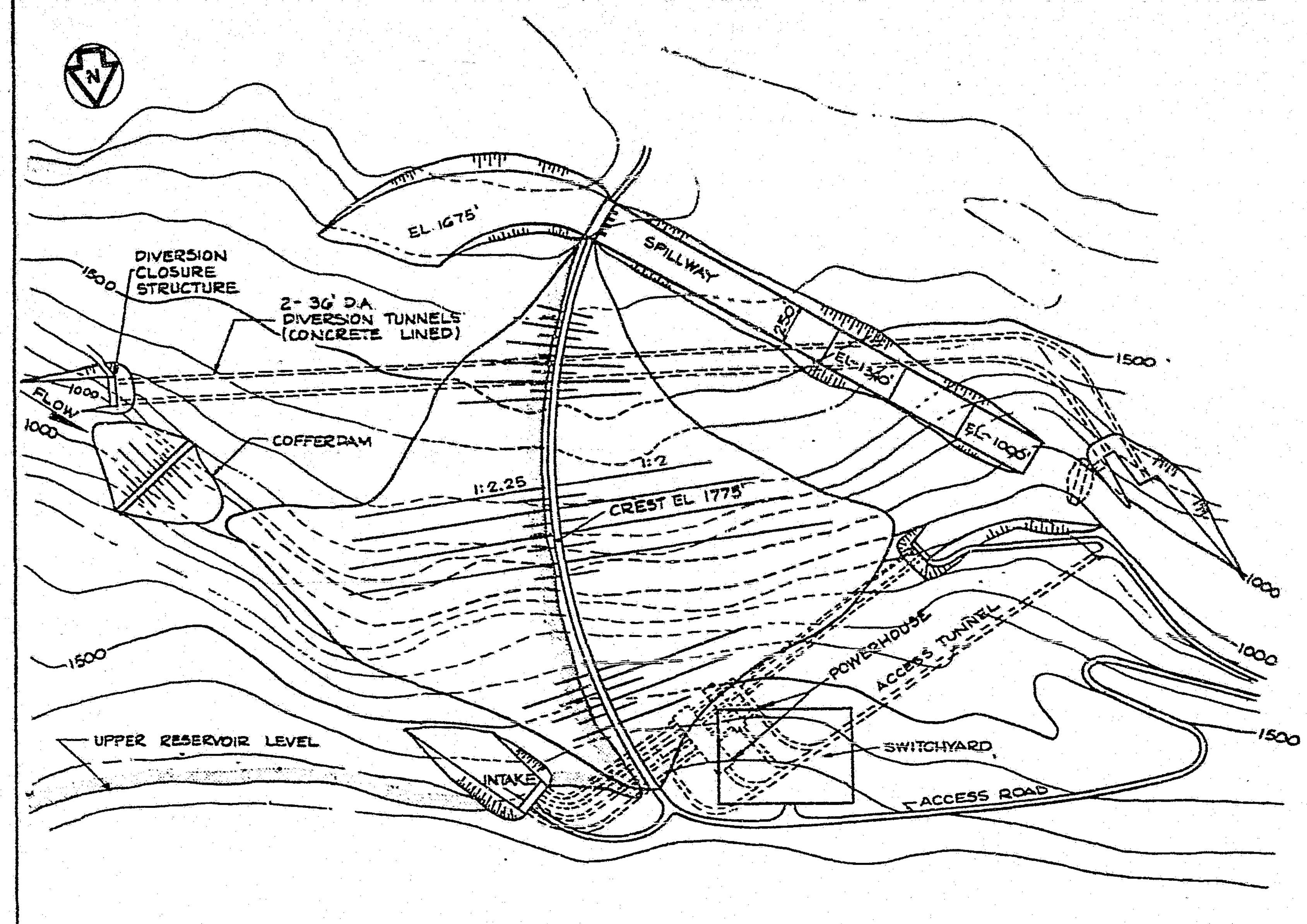
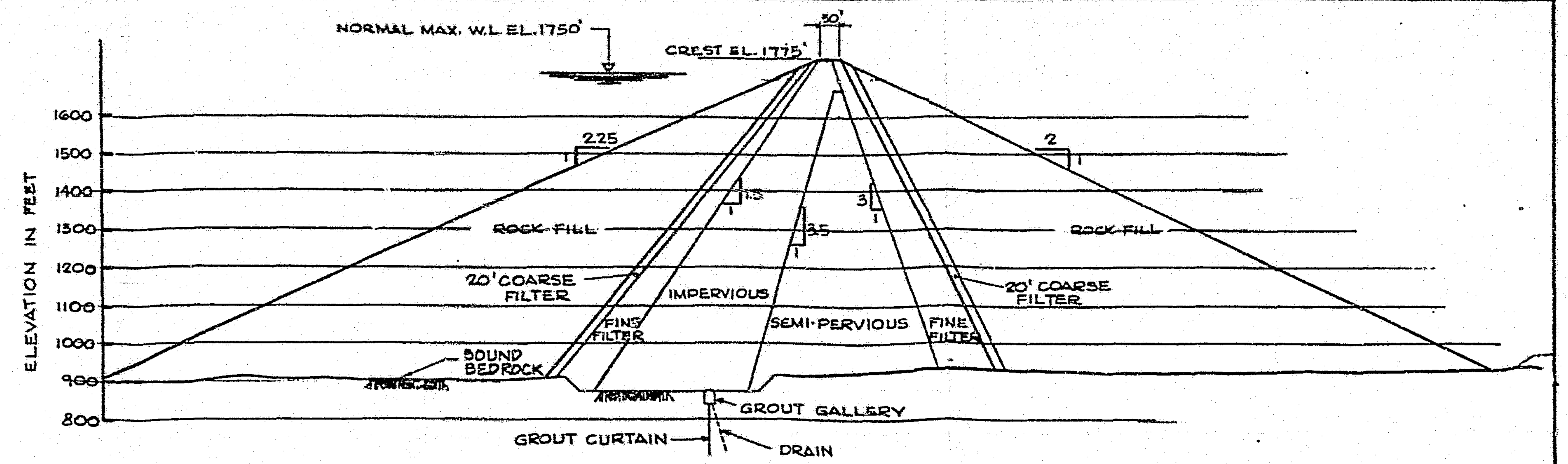

GENERAL ARRANGEMENT
SCALE : A
STAGE II INTAKE
STAGE I INTAKE
ACCESS TUNNEL
17' DIA CONCRETE LINED TUNNEL (STAGE II)
2-21' DIA CONCRETE LINED TUNNELS (STAGE I)
EL. 2060' STAGE I
EL. 2225' STAGE II
EXISTING GROUND
ORIGINAL BEDROCK
FLOW
STAGE I SPILLWAY
STAGE II SPILLWAY
35' DIA. TUNNELS
SECTION THRU DAM
SCALE : B
CREST EL. 2225'
NORMAL MAX. OPERATING EL. 2000'
EL. 2060'
STAGE II FILL
ROCK FILL
20' COARSE FILTER
IMPERVIOUS
FINE FILTER
SEMI-PERVIOUS
ROCK FILTER
GROUT CURTAIN
GROUT GALLERY
DRAIN
ELEVATION IN FEET
LONGITUDINAL SECTION THRU ℄ OF DAM
SCALE:B
CREST EL. 2225' AT ℄ OF DAM (STAGE II)
CREST EL 2060' AT ℄ OF DAM (STAGE I)
SLOPE
EXCAVATION FOR CORE
BEDROCK SURFACE
ORIGINAL GROUND SURFACE
GROUT GALLERIES
POWER FACILITIES PROFILE
SCALE : B
W.L. EL.2000'
EL 2014'
4-UNIT INTAKE (STAGE II)
EXISTING GROUND SURFACE
4-17' DIA. CONCRETE LINED TUNNELS
STAGE II CONCRETE PLUG
3-UNIT INTAKE (STAGE I)
3-17' DIA. CONCRETE LINED TUNNELS
TRANSFORMER AND DRAFT TUBE GATE GALLERY
POWERHOUSE
AVERAGE TAILWATER EL. 1465'
1-17' DIA TUNNEL (STAGE II)
MANIFOLD
CONSTRUCTION ADIT
GATE SHAFT
2-21' DIA TUNNELS (STAGE I)
SPILLWAY PROFILE
SCALE: B
NORMAL MAX. OPERATING LEVEL EL. 2200'
EXISTING GROUND SURFACE ON RIGHT SIDE OF SPILLWAY
EL. 2140'
EXISTING GROUND SURFACE ON ℄ OF SPILLWAY
EL. 2000'
WHEEL MOUNTED GATES
STAGE II SPILLWAY
SPILLWAY CONTROL STRUCTURE
2-35' DIA. CONCRETE LINED TUNNELS
EXISTING GROUND SURFACE ON LEFT SIDE OF SPILLWAY
STAGE I SPILLWAY
AVERAGE TAILWATER EL. 1465'
CHAINAGE IN FEET
SCALE A 0 500 1000 FEET
SCALE B 0 200 400 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA
STAGED FILL DAM
DATE DEC. 1981
SCALE AS SHOWN
SK-5700-C6-104
DATE NO. REVISIONS CK. APP. APP.

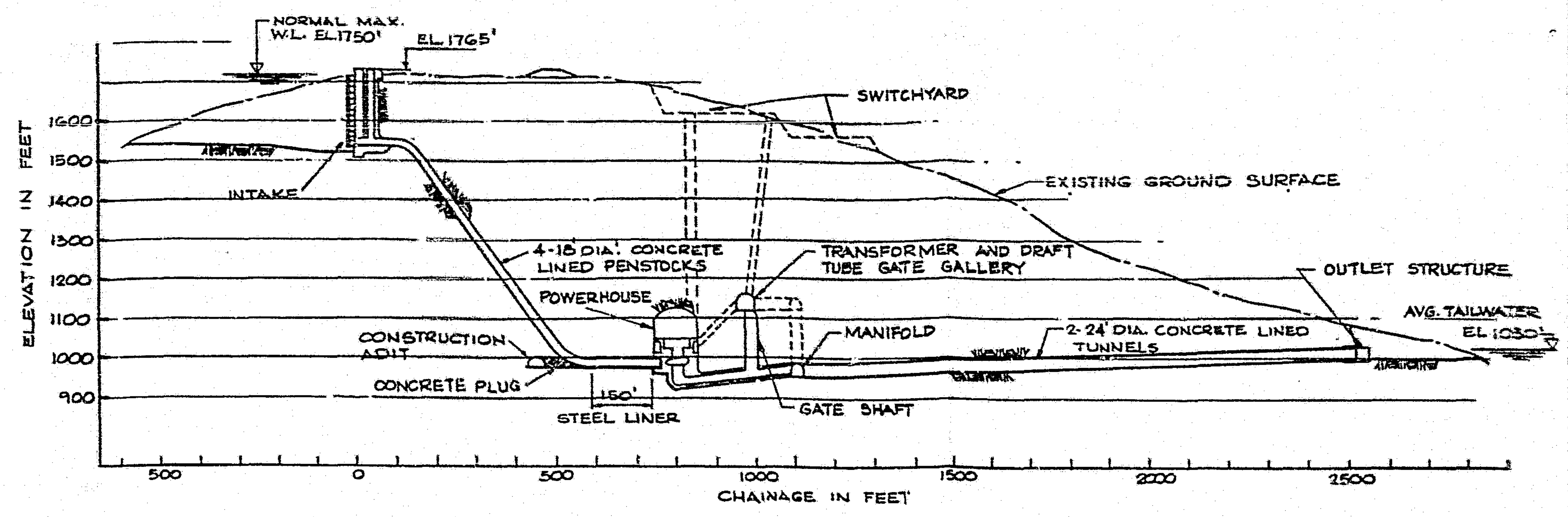
FIGURE B.8



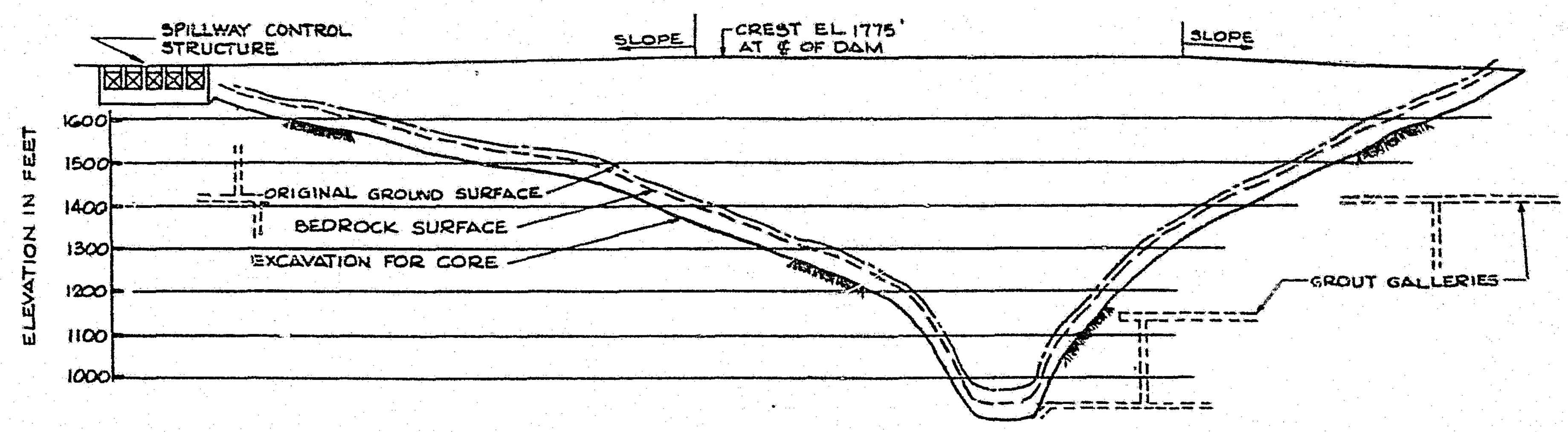
GENERAL ARRANGEMENT
SCALE : A



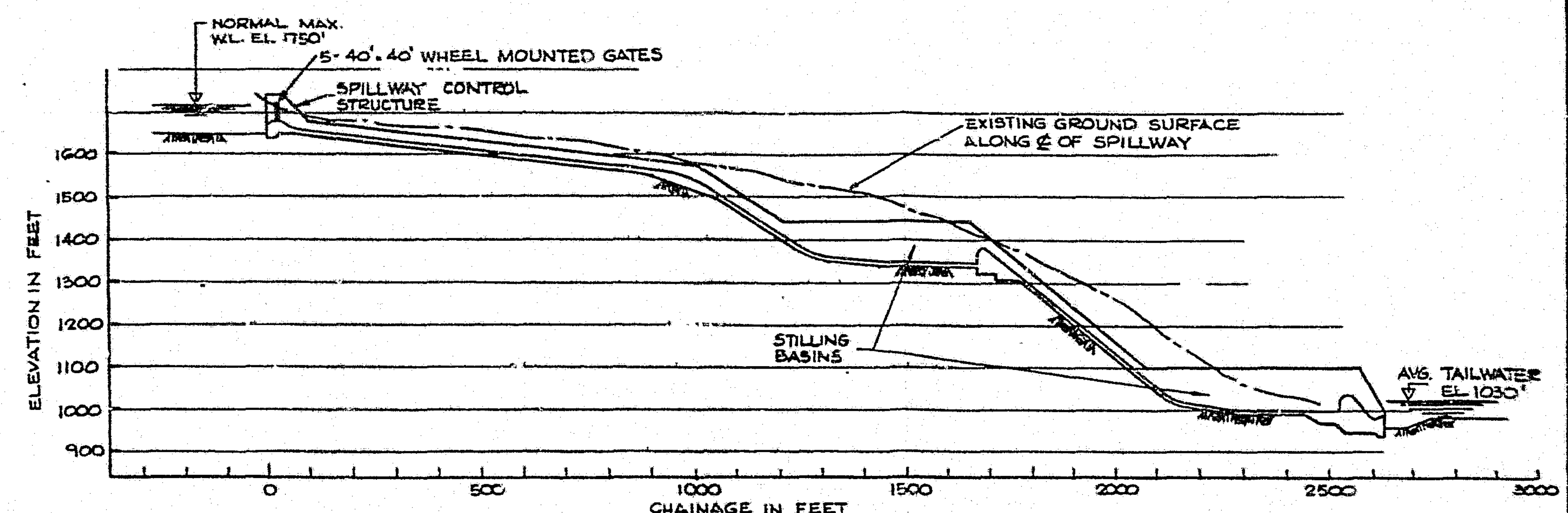
SECTION THRU DAM
SCALE: B



POWER FACILITIES PROFILE
SCALE: B



LONGITUDINAL SECTION THRU ℄ OF DAM
SCALE: B



SPILLWAY PROFILE
SCALE: B

SCALE A 0 400 800 FEET

SCALE B 0 200 400 FEET

FIGURE B.9

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

HIGH DEVIL CANYON
HYDRO DEVELOPMENT

DATE DEC. 1981 | SCALE AS SHOWN | DRAWING NO. SK-5700-C8-101

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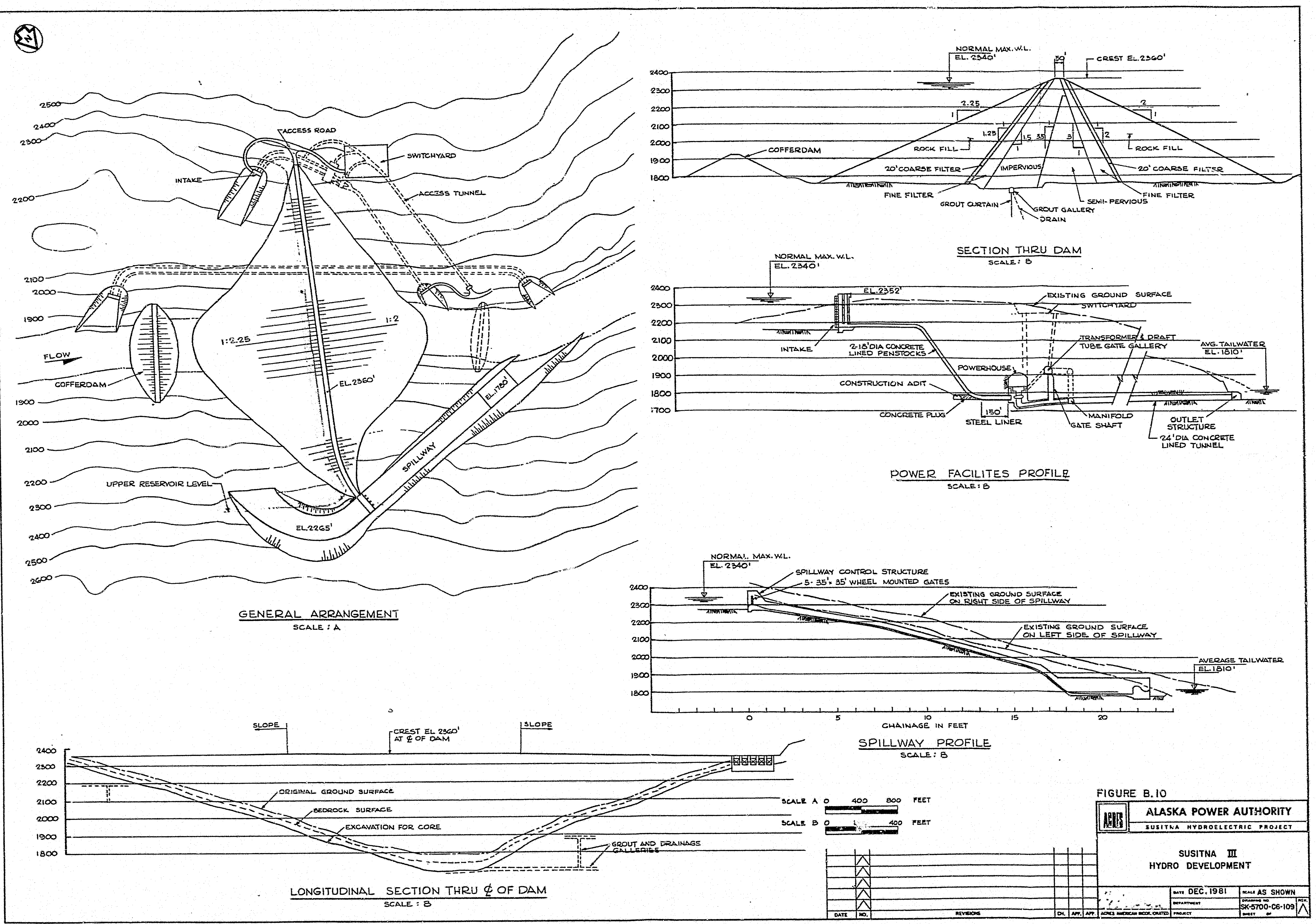
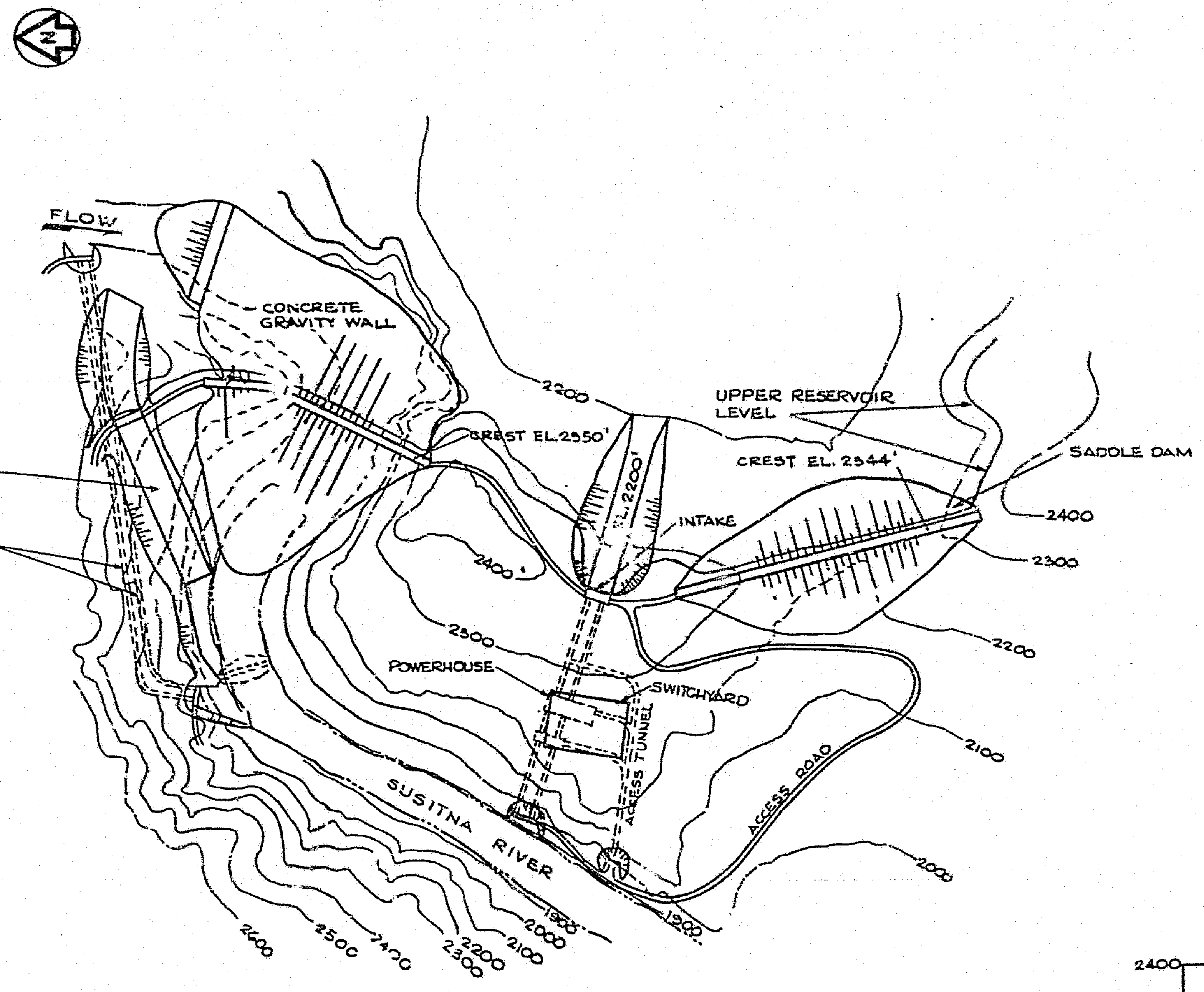
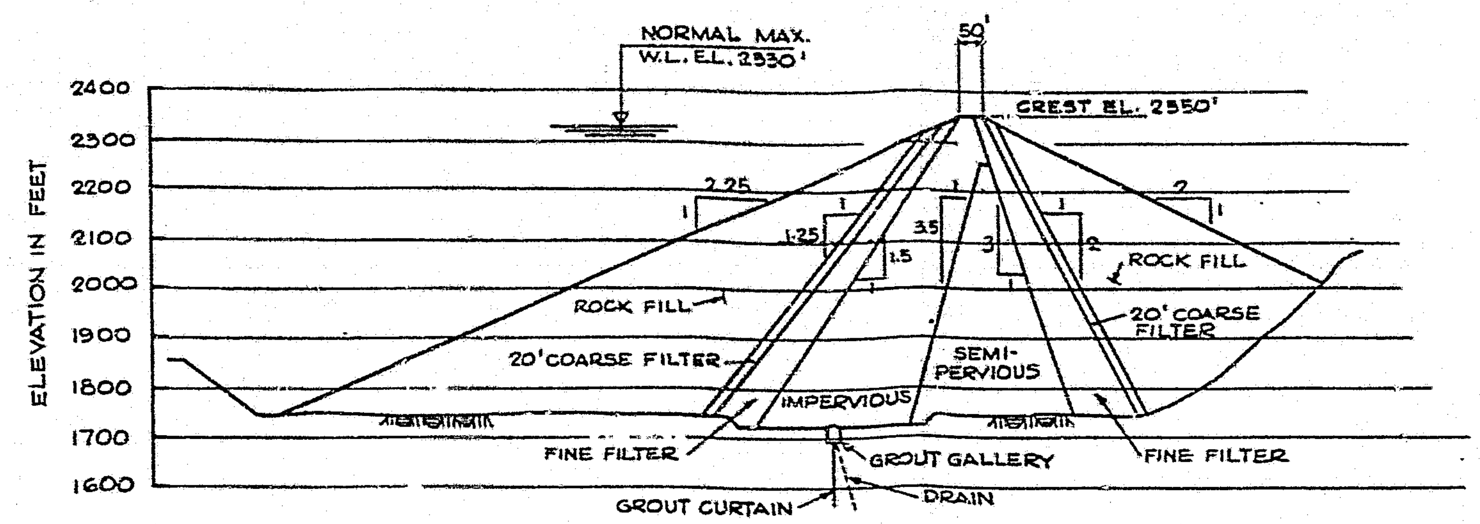
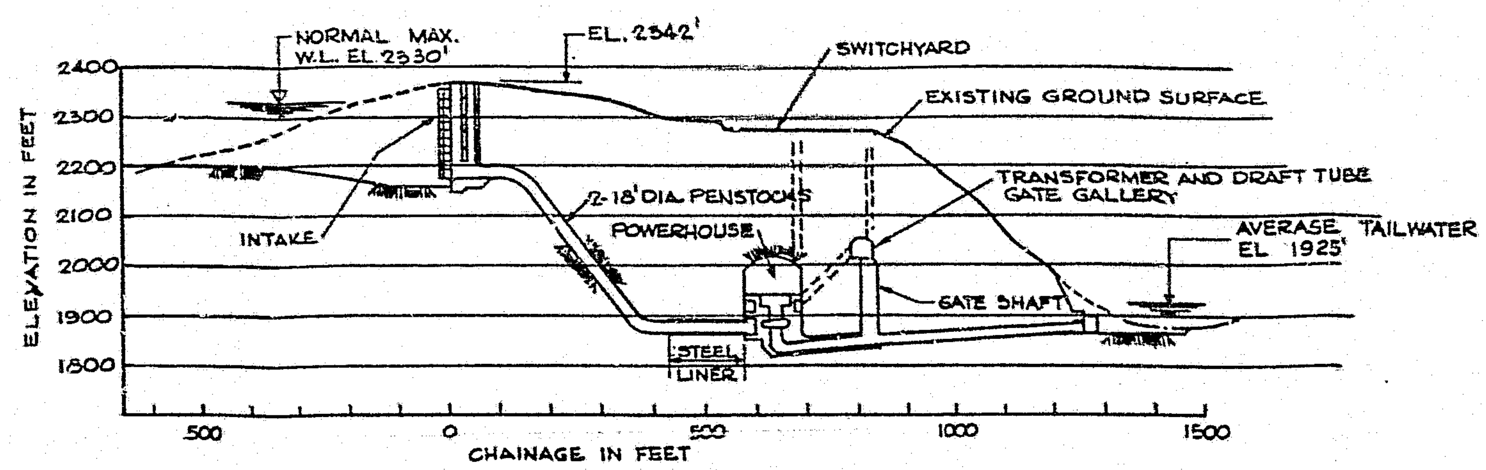

ACCESS ROAD
SWITCHYARD
INTAKE
ACCESS TUNNEL
FLOW
COFFERDAM
1:2.25
1:2
EL.2360'
EL.1780'
SPILLWAY
UPPER RESERVOIR LEVEL
EL.2265'
GENERAL ARRANGEMENT
SCALE : A
NORMAL MAX. W.L.
EL. 2340'
CREST EL.2360'
30'
COFFERDAM
ROCK FILL
20' COARSE FILTER
IMPERVIOUS
FINE FILTER
GROUT CURTAIN
GROUT GALLERY
DRAIN
SEMI-PERVIOUS
SECTION THRU DAM
SCALE : B
EL.2352'
EXISTING GROUND SURFACE
SWITCHYARD
INTAKE
2-18' DIA CONCRETE LINED PENSTOCKS
TRANSFORMER & DRAFT TUBE GATE GALLERY
AVG. TAILWATER EL. 1810'
POWERHOUSE
CONSTRUCTION ADIT
CONCRETE PLUG
150'
STEEL LINER
MANIFOLD
GATE SHAFT
OUTLET STRUCTURE
24' DIA CONCRETE LINED TUNNEL
POWER FACILITES PROFILE
SCALE : B
SPILLWAY CONTROL STRUCTURE
5 - 35' x 35' WHEEL MOUNTED GATES
EXISTING GROUND SURFACE ON RIGHT SIDE OF SPILLWAY
EXISTING GROUND SURFACE ON LEFT SIDE OF SPILLWAY
AVERAGE TAILWATER EL. 1810'
CHAINAGE IN FEET
SPILLWAY PROFILE
SCALE : B
SLOPE
CREST EL 2360' AT ℄ OF DAM
ORIGINAL GROUND SURFACE
BEDROCK SURFACE
EXCAVATION FOR CORE
GROUT AND DRAINAGE GALLERIES
LONGITUDINAL SECTION THRU ℄ OF DAM
SCALE : B
SCALE A 0 400 800 FEET
SCALE B 0 400 FEET
DATE
NO.
REVISIONS
FIGURE B.10
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
SUSITNA III
HYDRO DEVELOPMENT
DEC. 1981
AS SHOWN
SK-5700-C6-109



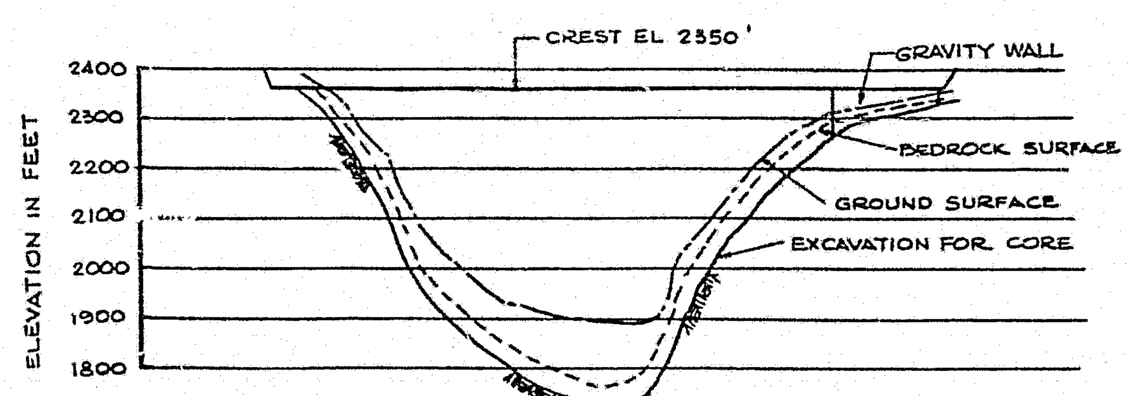
GENERAL ARRANGEMENT
SCALE: A



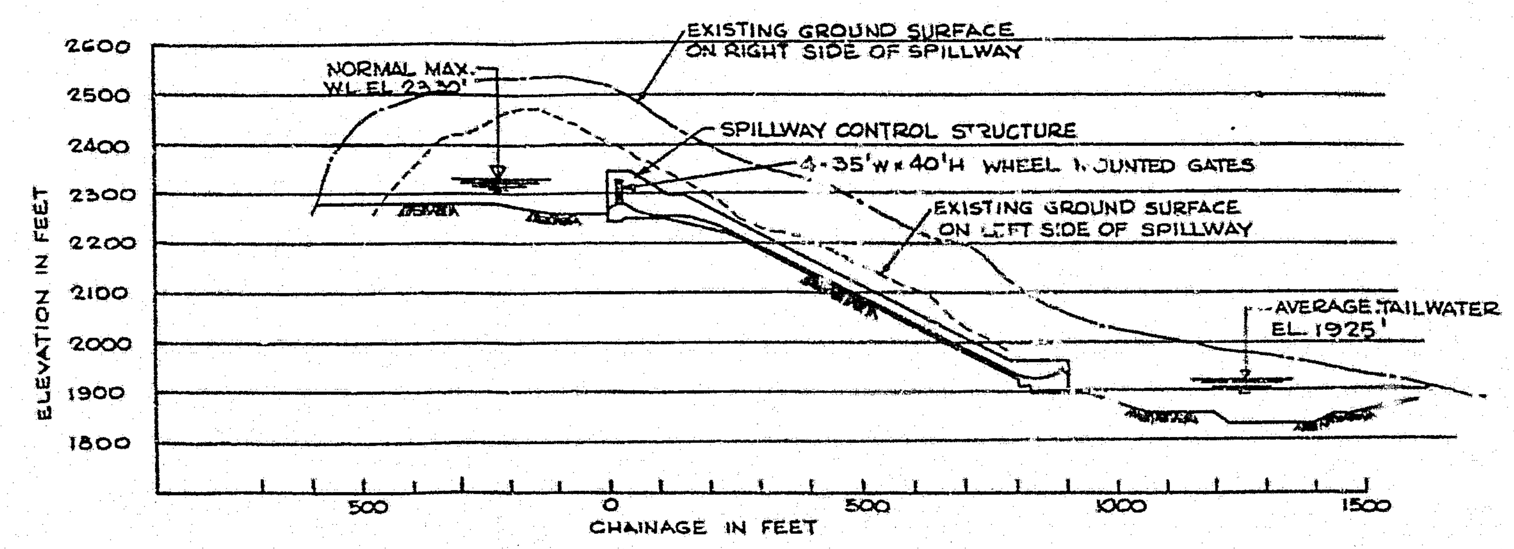
SECTION THRU MAIN DAM
SCALE: B



POWER FACILITIES PROFILE
SCALE: B



LONGITUDINAL SECTION THRU ℄ OF MAIN DAM
SCALE: B



SPILLWAY PROFILE
SCALE: B

SCALE A 0 400 800 FEET
SCALE B 0 200 400 FEET

FIGURE B.11

DATE	NO.	REVISIONS	CH.	APP.	APP.

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ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

VEE
HYDRO DEVELOPMENT

DATE DEC.1981
SCALE AS SHOWN
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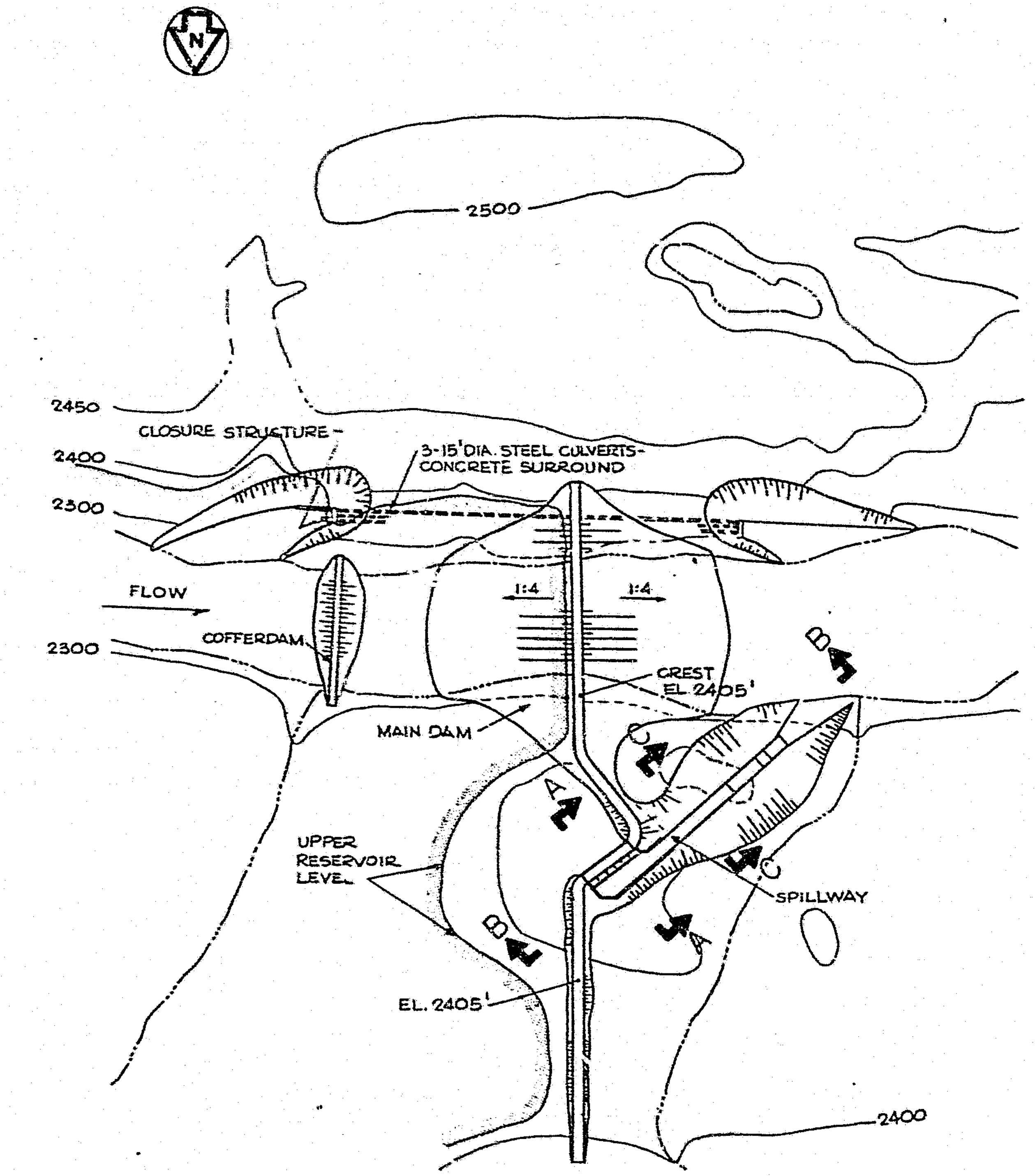
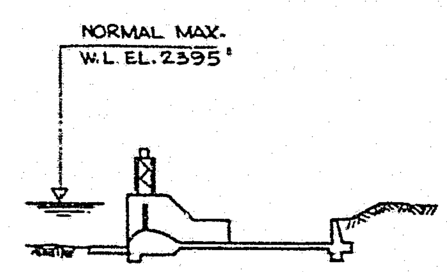
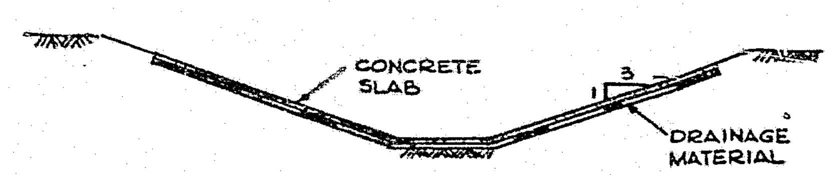
DRAWING NO. SK-5700-C6-107



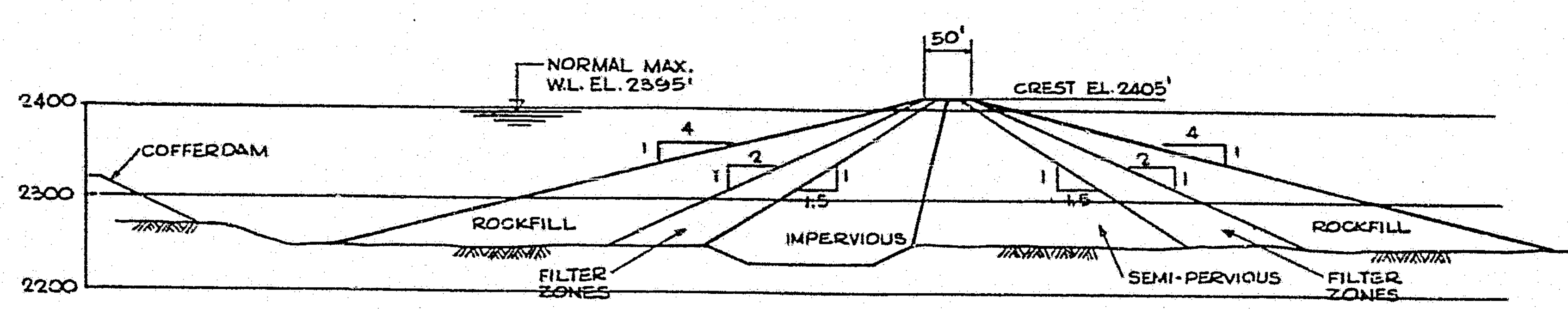
MACLAREN
GENERAL ARRANGEMENT
SCALE: A



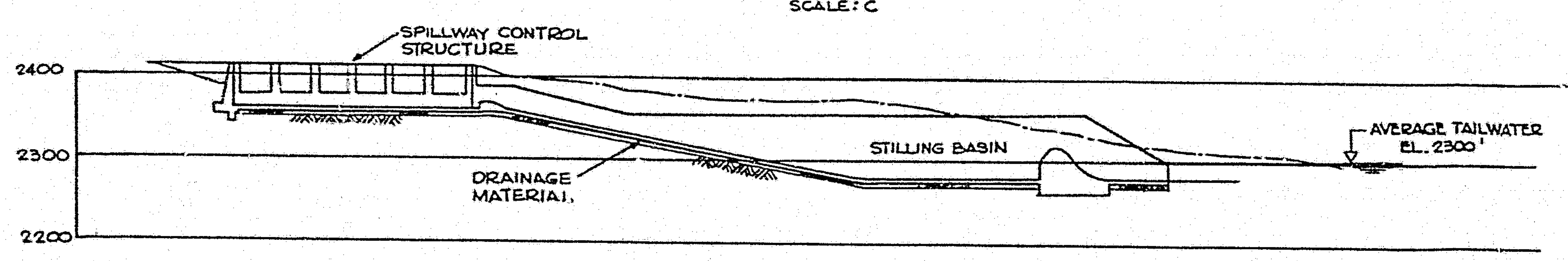
SECTION A-A



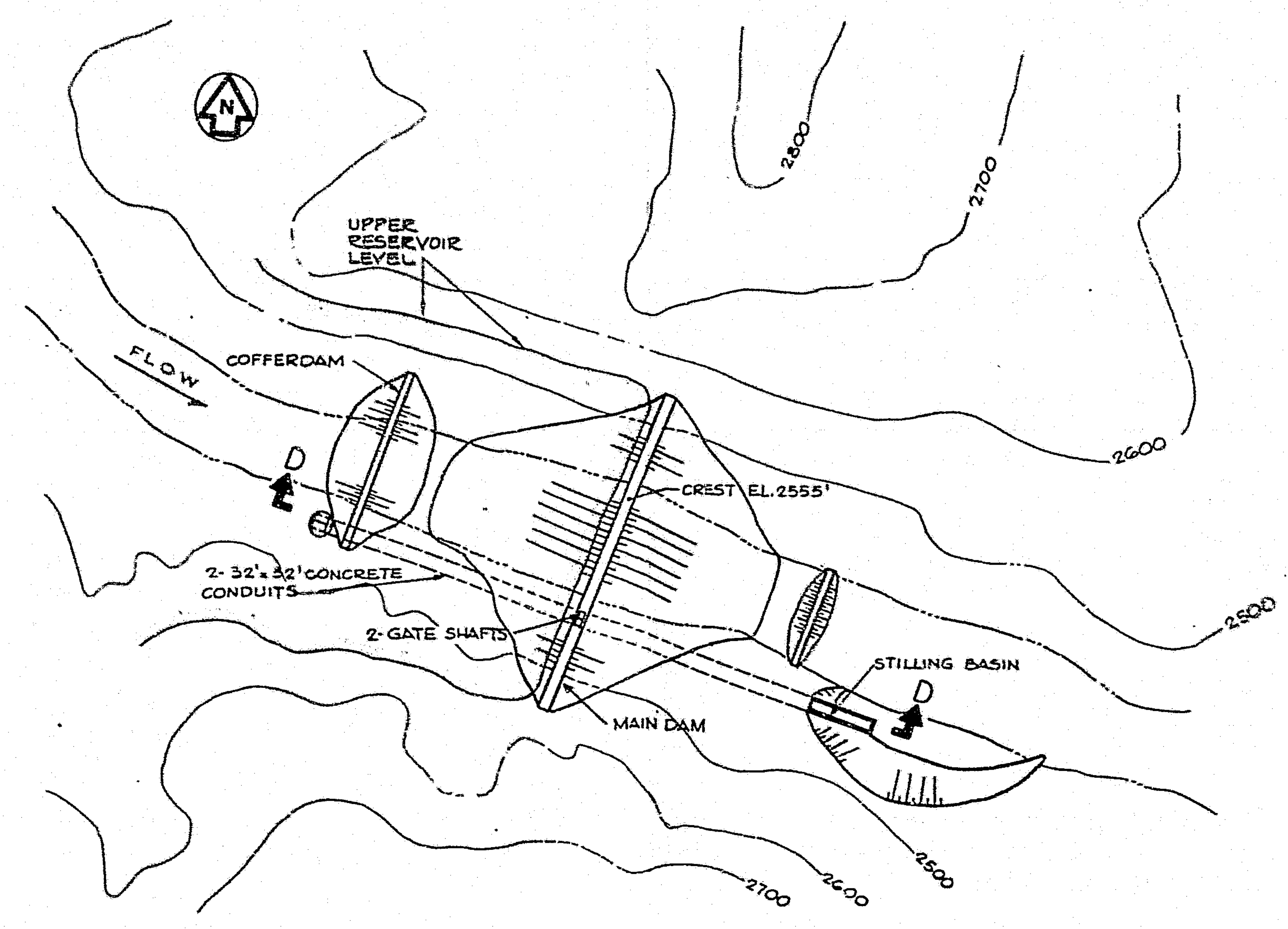
SECTION C-C



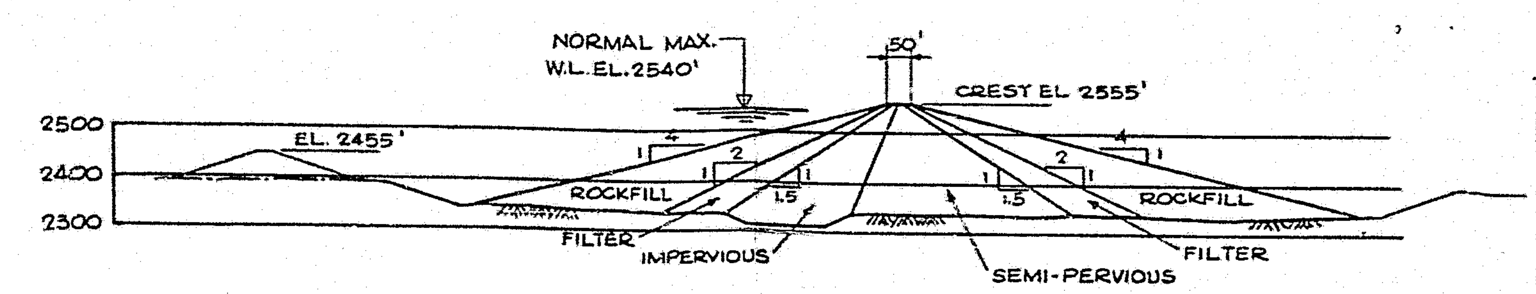
DAM CROSS SECTION
SCALE: C



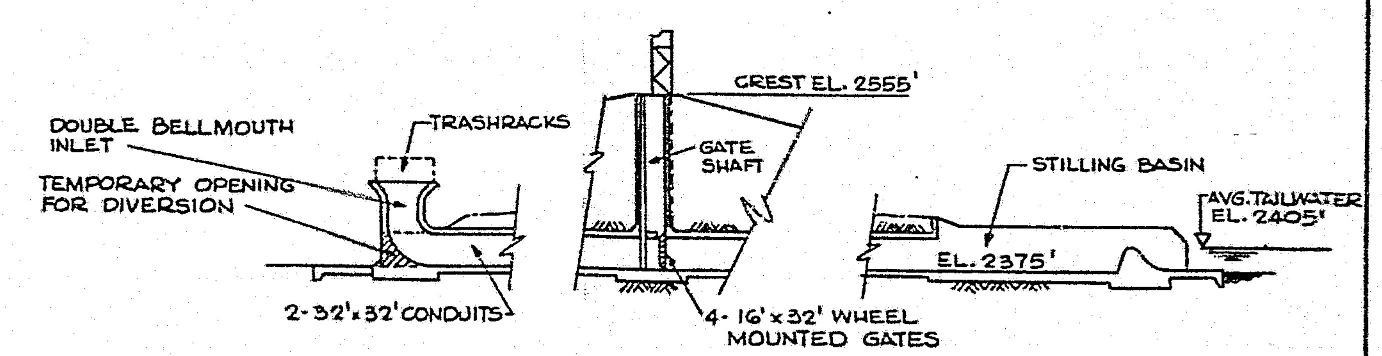
SECTION B-B
SCALE: C



DENALI
GENERAL ARRANGEMENT
SCALE: A



DAM CROSS SECTION
SCALE: B



SECTION D-D
SCALE: C

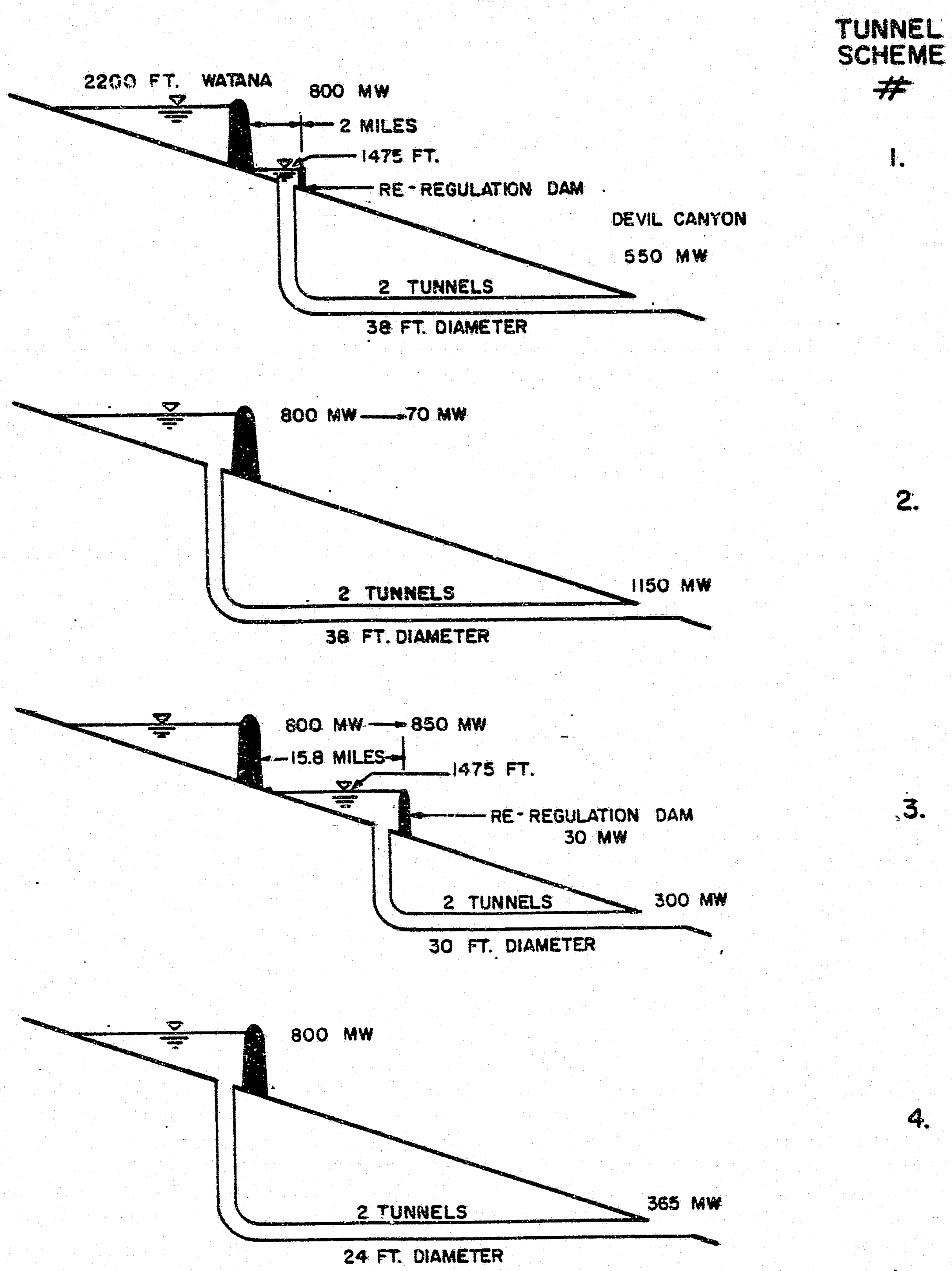
SCALE A 0 400 800 FEET
SCALE B 0 200 400 FEET
SCALE C 0 100 200 FEET

FIGURE B.12

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

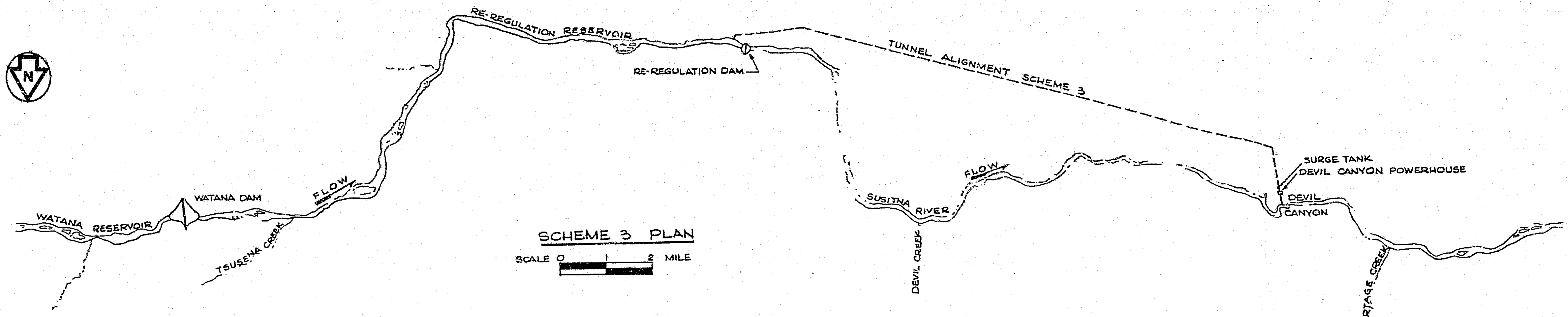
DENALI & MACLAREN
HYDRO DEVELOPMENTS

DATE DEC. 1981
SCALE AS SHOWN
SK-5700-C6-108
ACRES AMERICAN INCORPORATED



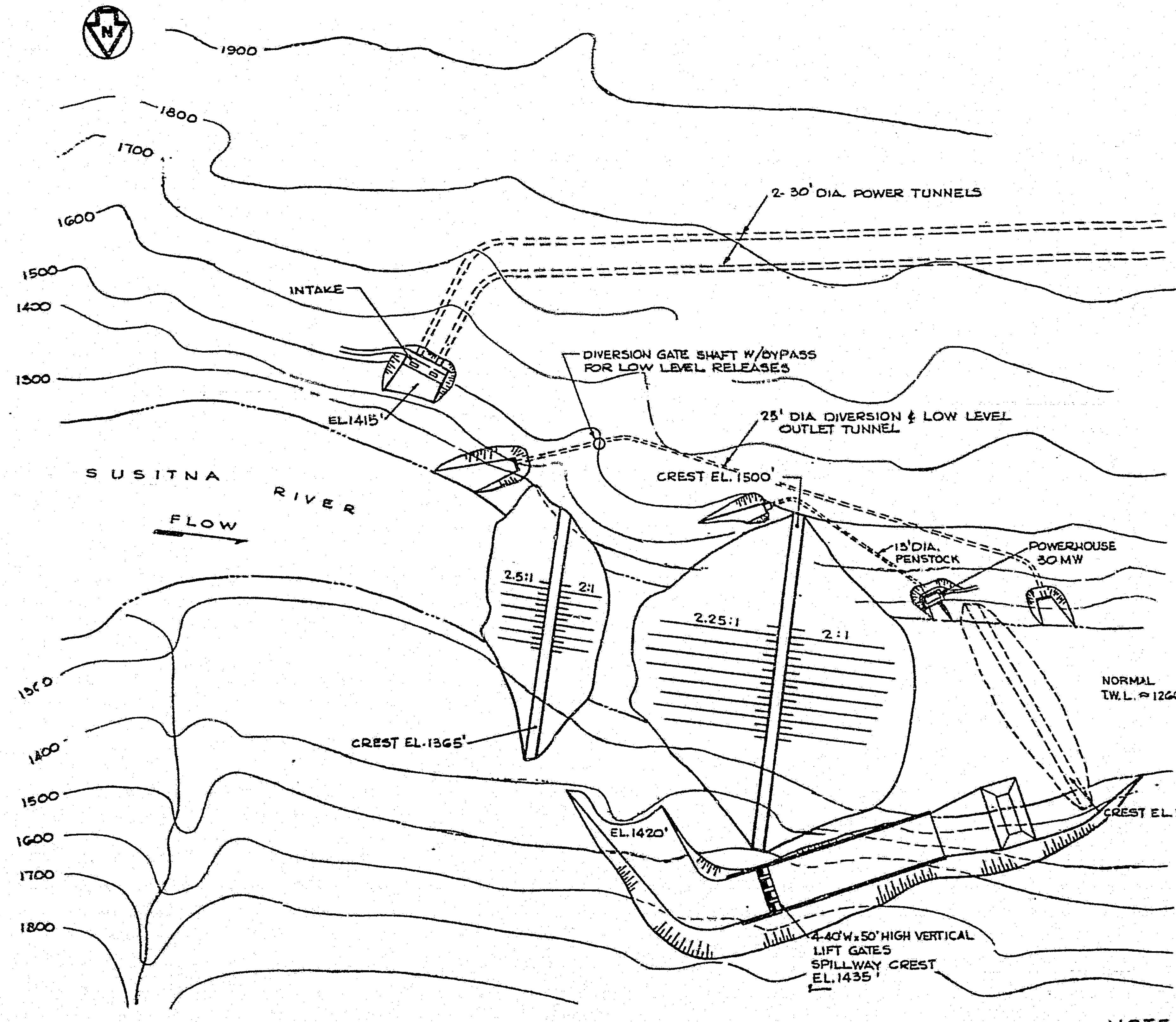
SCHEMATIC REPRESENTATION
OF CONCEPTUAL TUNNEL SCHEMES

FIGURE B.13



SCHEME 3 PLAN

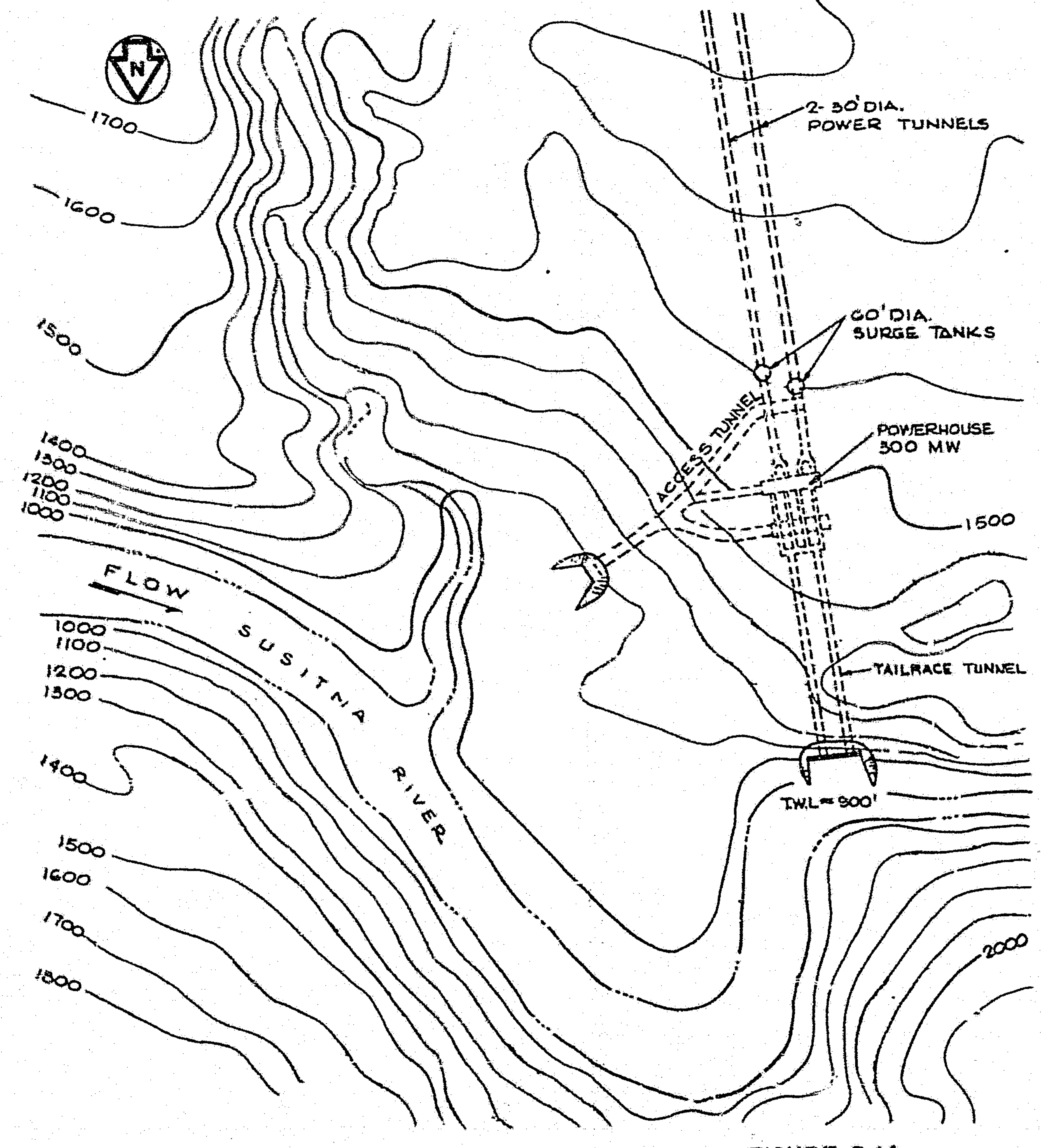
SCALE 0 1 2 MILE



GENERAL ARRANGEMENT
RE-REGULATION DAM

SCALE 0 300 600 FEET

NOTE:
ALL PLANS AND LAYOUTS FOR CONCEPTUAL STUDY PURPOSES ONLY.



GENERAL ARRANGEMENT
DEVIL CANYON POWERHOUSE

SCALE 0 400 800 FEET

FIGURE B.14

DATE	NO.	REVISIONS	CH.	APP.	APP.

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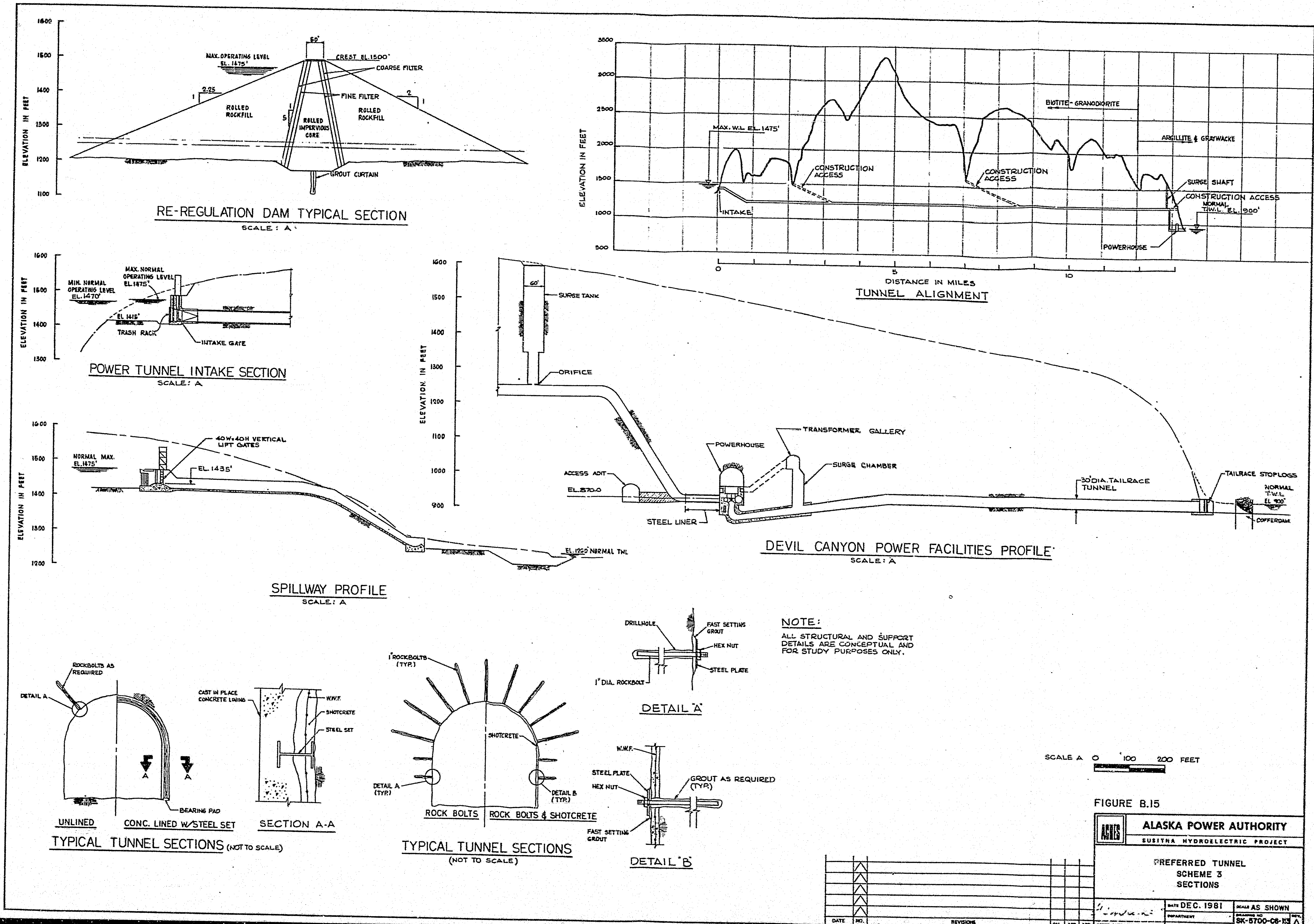
SUSITNA HYDROELECTRIC PROJECT

PREFERRED TUNNEL
SCHEME 3
PLAN VIEWS

ACRES AMERICAN INCORPORATED

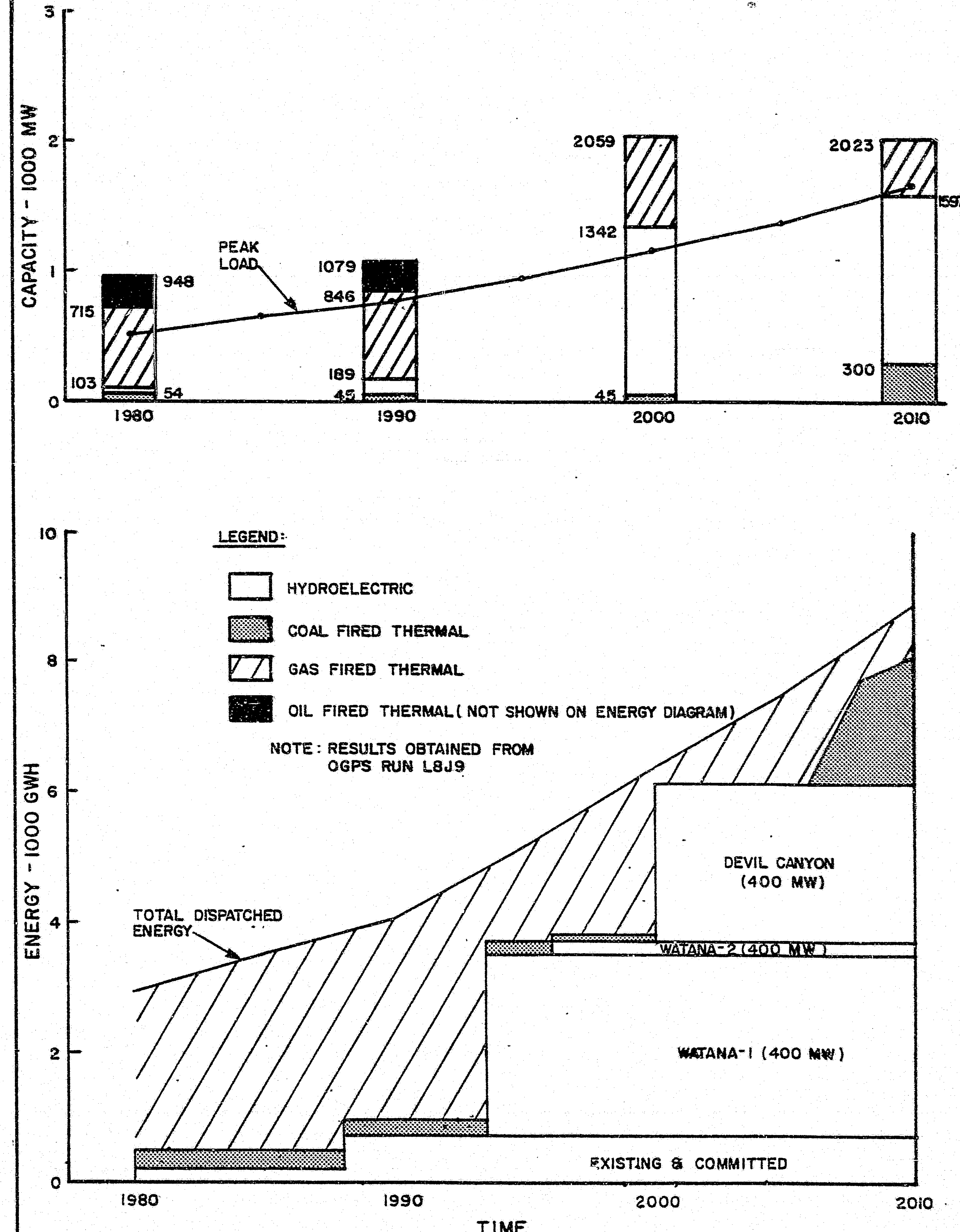
DATE DEC. 1981 — SCALE AS SHOWN

DRAWING NO. SK-8700-C6-82



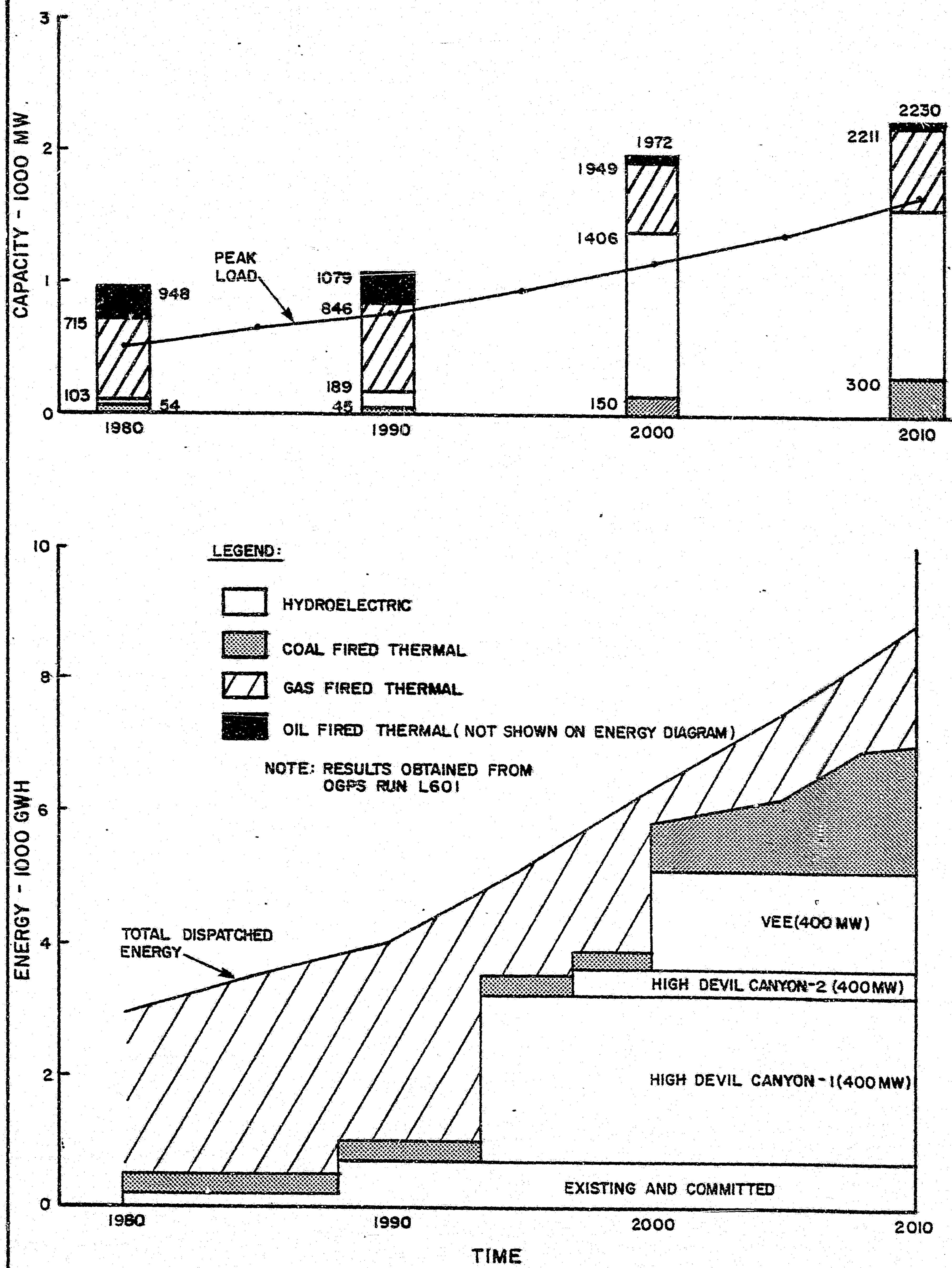
RE-REGULATION DAM TYPICAL SECTION
SCALE: A
MAX. OPERATING LEVEL EL. 1475'
CREST EL. 1500'
50'
COARSE FILTER
FINE FILTER
ROLLED ROCKFILL
ROLLED IMPERVIOUS CORE
GROUT CURTAIN
2.25
ELEVATION IN FEET
TUNNEL ALIGNMENT
DISTANCE IN MILES
MAX. W.L. EL. 1475'
CONSTRUCTION ACCESS
BIOTITE - GRANODIORITE
ARGILLITE & GRAYWACKE
SURGE SHAFT
NORMAL T.W.L. EL. 900'
INTAKE
POWERHOUSE
POWER TUNNEL INTAKE SECTION
SCALE: A
MIN. NORMAL OPERATING LEVEL EL. 1470'
MAX. NORMAL OPERATING LEVEL EL. 1475'
EL. 1415'
TRASH RACK
INTAKE GATE
DEVIL CANYON POWER FACILITIES PROFILE
SCALE: A
SURGE TANK
ORIFICE
TRANSFORMER GALLERY
SURGE CHAMBER
ACCESS ADIT
EL. 870.0
STEEL LINER
30' DIA. TAILRACE TUNNEL
TAILRACE STOPLOGS
COFFERDAM
SPILLWAY PROFILE
SCALE: A
NORMAL MAX. EL. 1475'
40W x 40H VERTICAL LIFT GATES
EL. 1435'
EL. 1220' NORMAL TWL
DRILLHOLE
FAST SETTING GROUT
HEX NUT
STEEL PLATE
1" DIA. ROCKBOLT
DETAIL "A"
NOTE:
ALL STRUCTURAL AND SUPPORT DETAILS ARE CONCEPTUAL AND FOR STUDY PURPOSES ONLY.
ROCKBOLTS AS REQUIRED
DETAIL A
CAST IN PLACE CONCRETE LINING
W.W.F.
SHOTCRETE
STEEL SET
BEARING PAD
UNLINED
CONC. LINED W/STEEL SET
SECTION A-A
TYPICAL TUNNEL SECTIONS (NOT TO SCALE)
1" ROCKBOLTS (TYP.)
DETAIL A (TYP.)
DETAIL B (TYP.)
ROCK BOLTS
ROCK BOLTS & SHOTCRETE
TYPICAL TUNNEL SECTIONS
(NOT TO SCALE)
GROUT AS REQUIRED (TYP.)
DETAIL "B"
SCALE A 0 100 200 FEET
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
PREFERRED TUNNEL SCHEME 3 SECTIONS
DEC. 1981
AS SHOWN
SK-5700-C6-K3
REVISIONS

FIGURE B.15



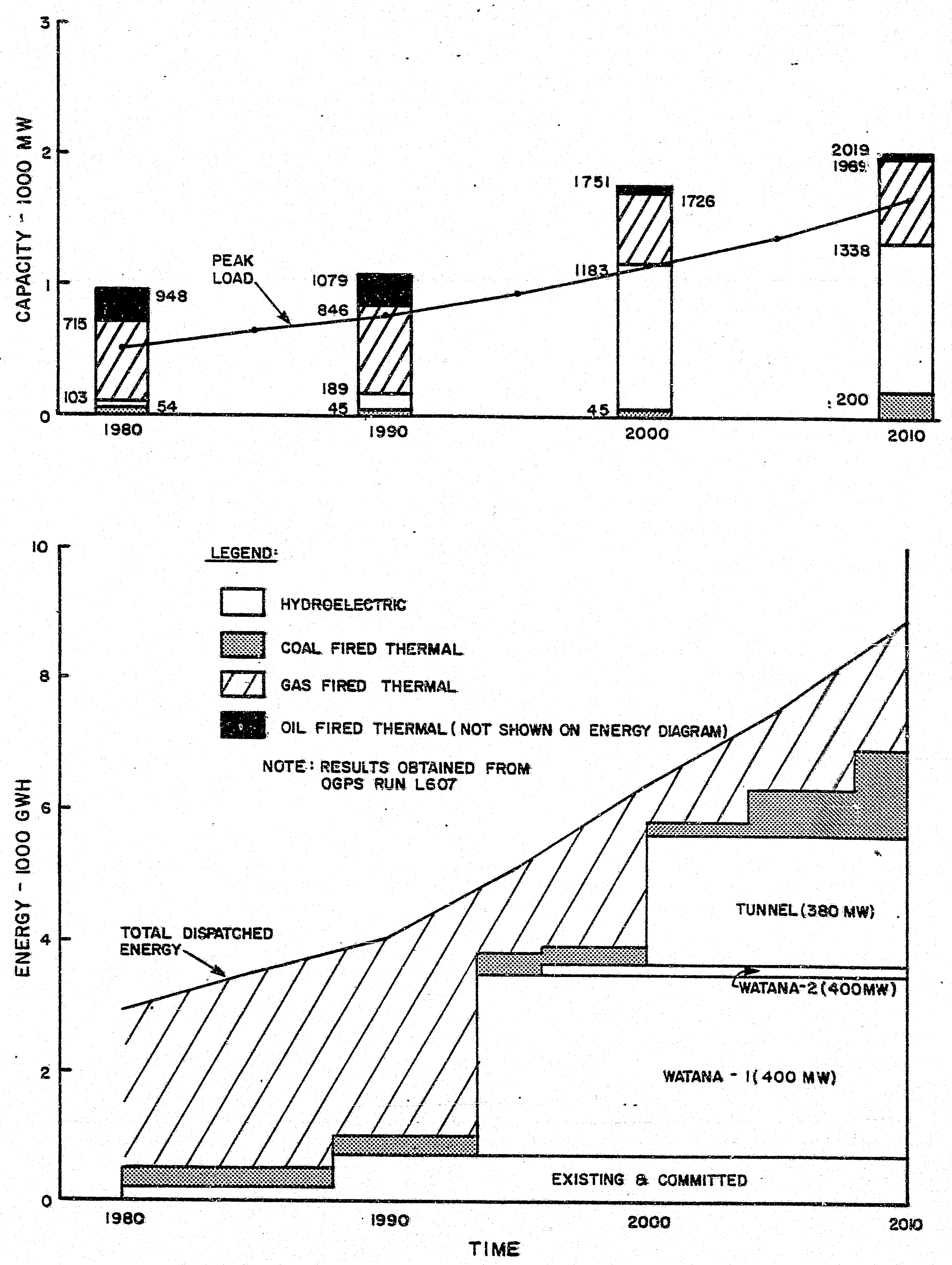
GENERATION SCENARIO WITH SUSITNA PLAN E1.3
-MEDIUM LOAD FORECAST-

FIGURE B.16



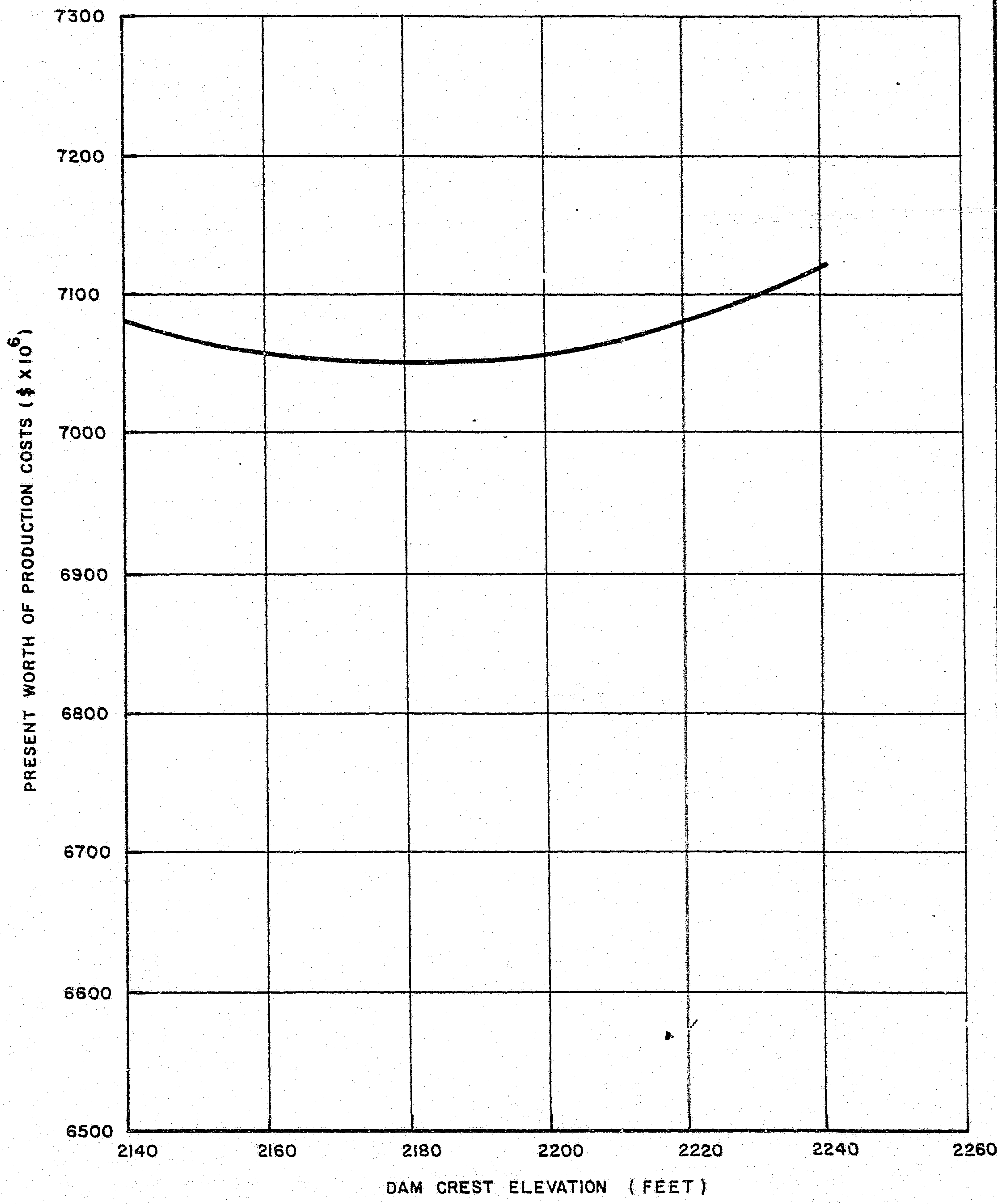
GENERATION SCENARIO WITH SUSITNA PLAN E 2.3
-MEDIUM LOAD FORECAST-

FIGURE B.17



GENERATION SCENARIO WITH SUSITNA PLAN E3.1
-MEDIUM LOAD FORECAST-

FIGURE B.18

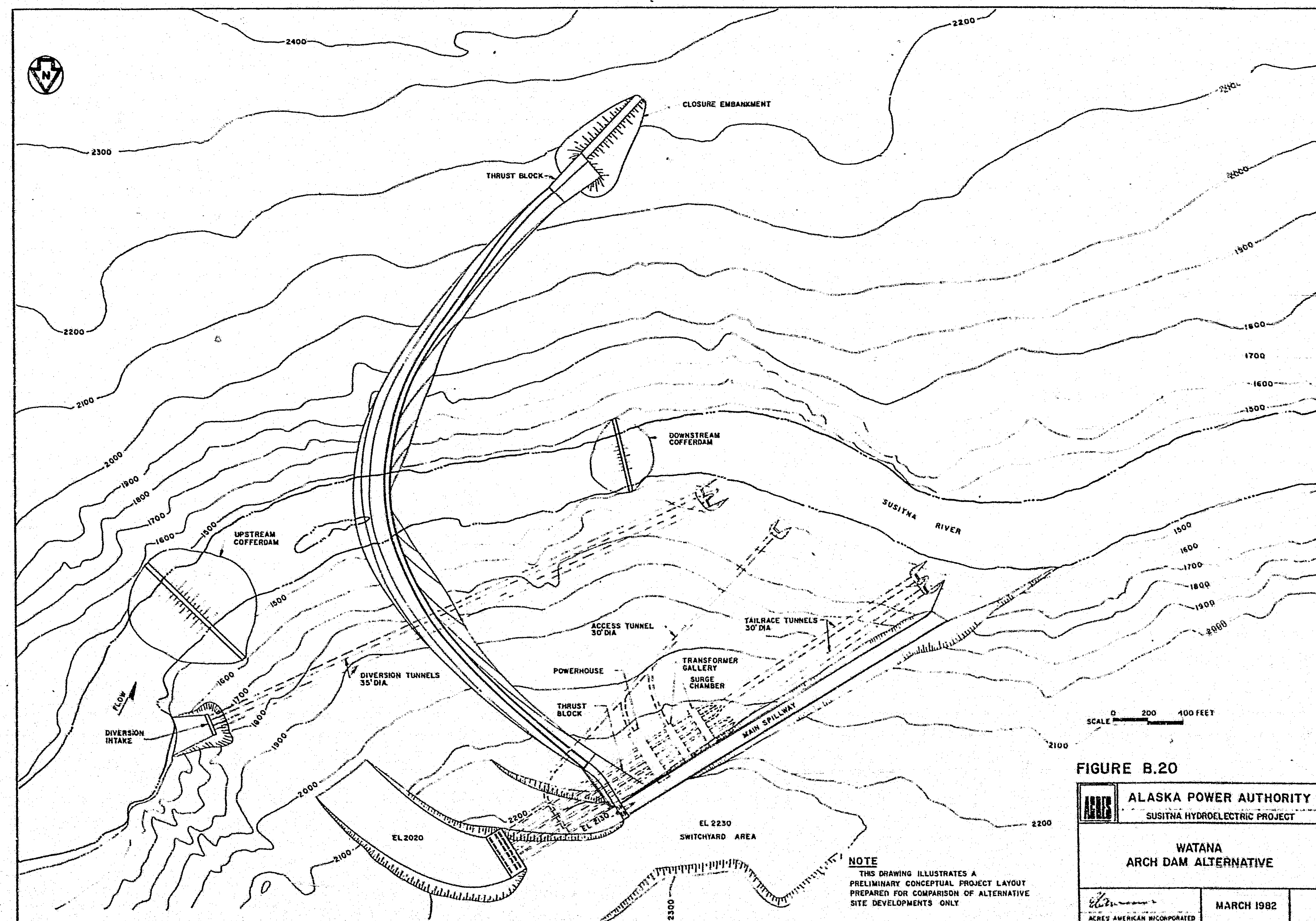


WATANA RESERVOIR

DAM CREST ELEVATION / PRESENT WORTH OF PRODUCTION COSTS

FIGURE B.19

ACRES



NOTE
THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY.

FIGURE B.20

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

WATANA
ARCH DAM ALTERNATIVE

ACRES AMERICAN INCORPORATED

MARCH 1982

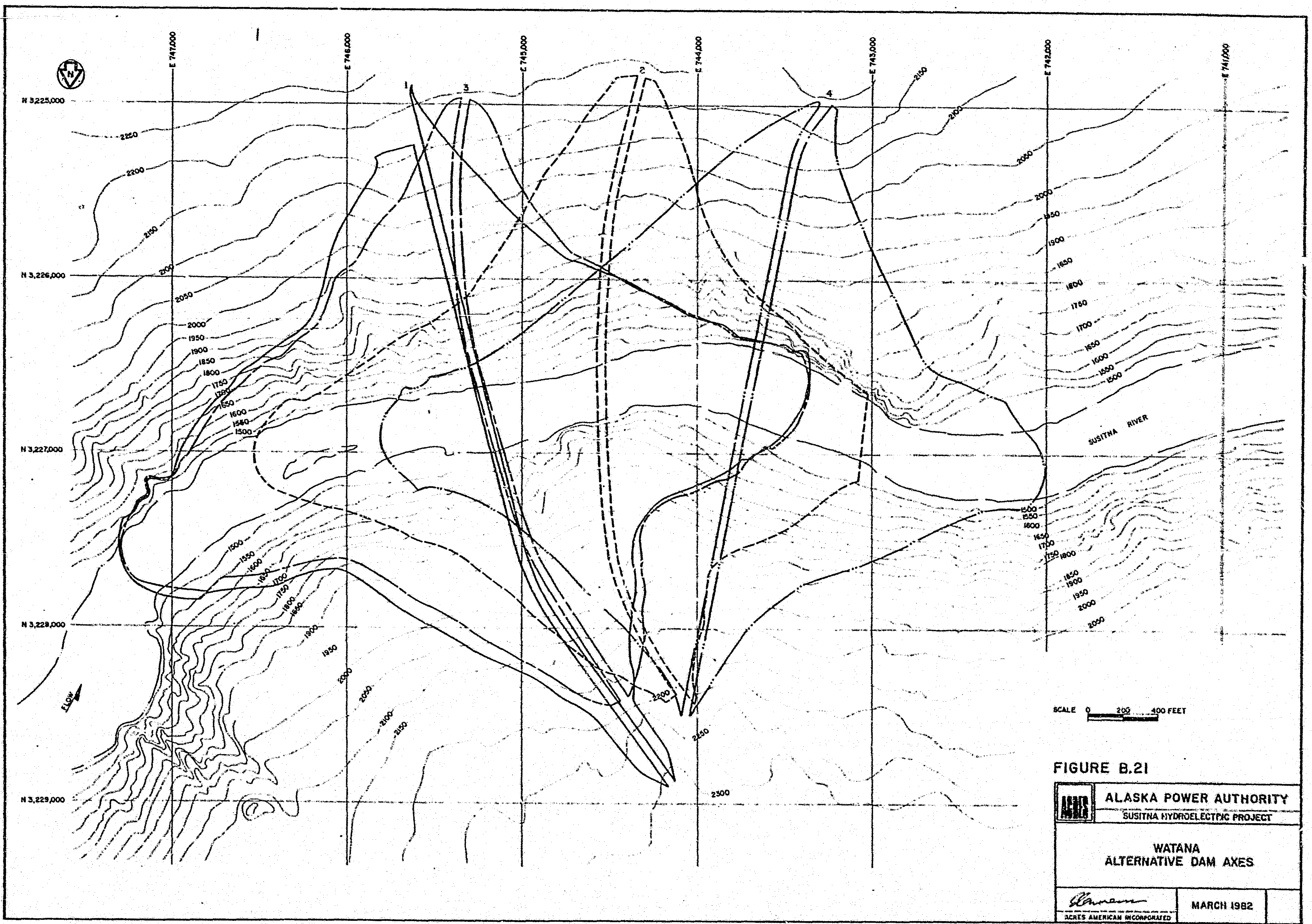


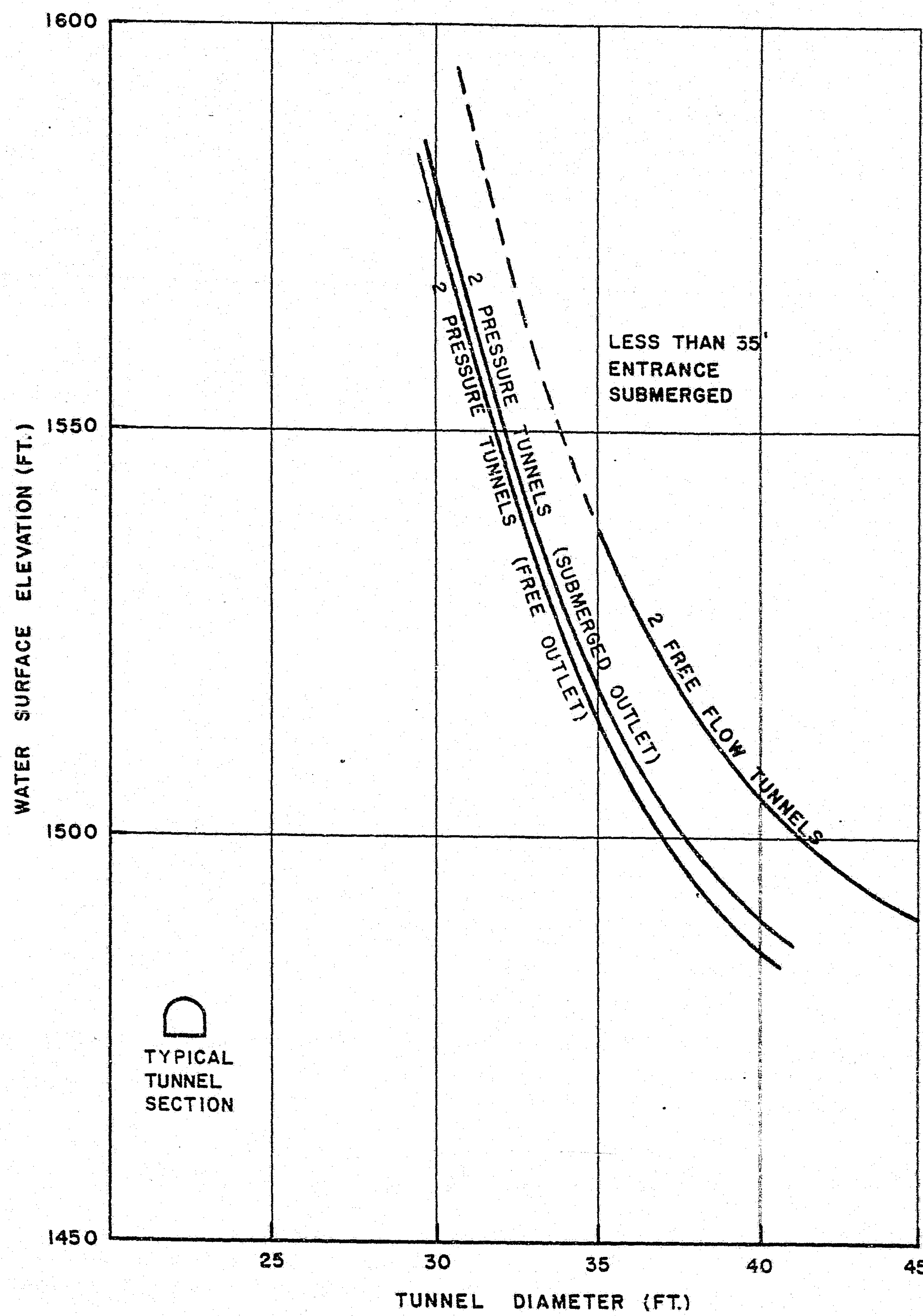
FIGURE B.21

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

WATANA
ALTERNATIVE DAM AXES

ACRES AMERICAN INCORPORATED

MARCH 1982

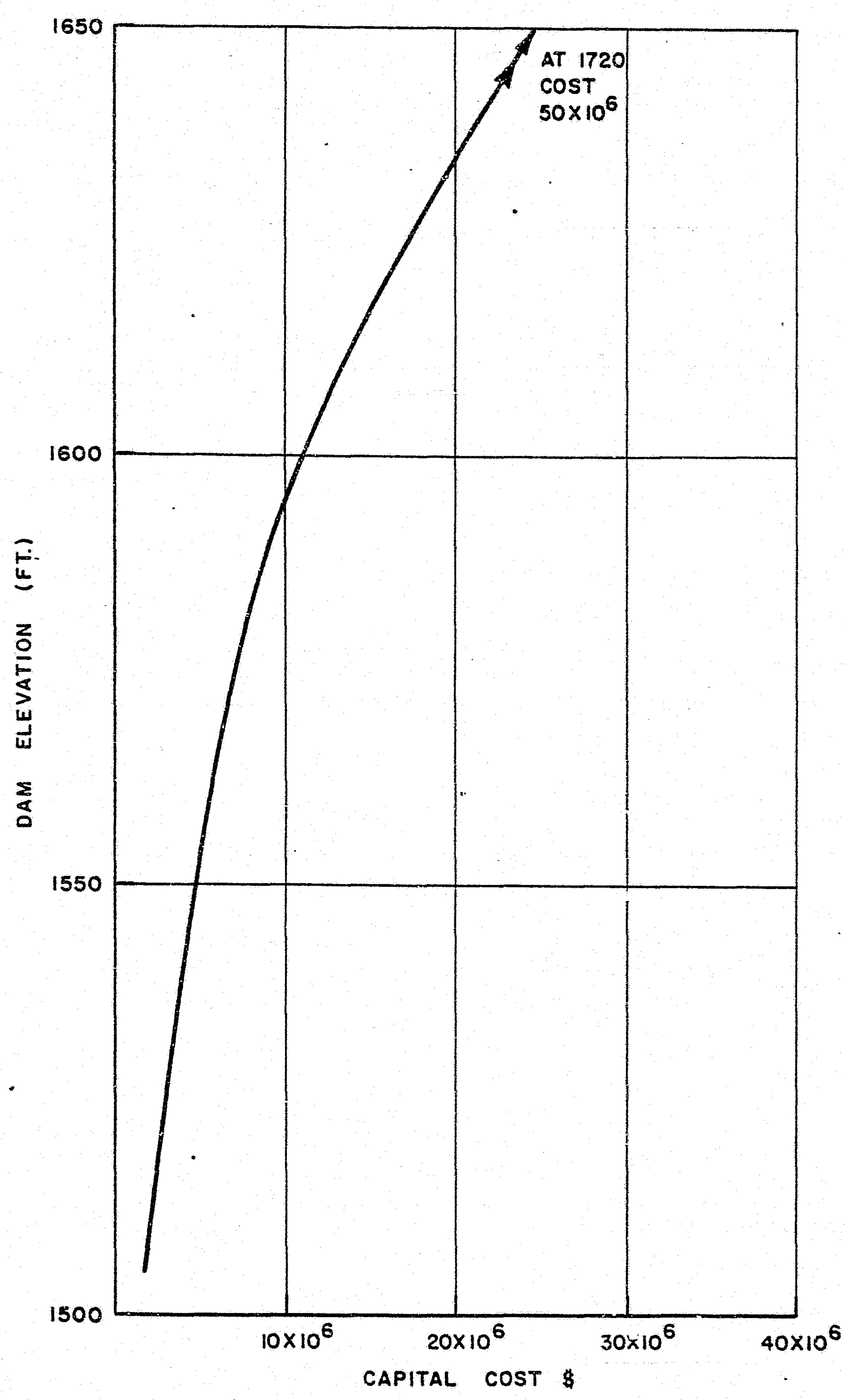


NOTE
FOR 80,000 CFS

WATANA DIVERSION
HEADWATER ELEVATION / TUNNEL DIAMETER

ACRES

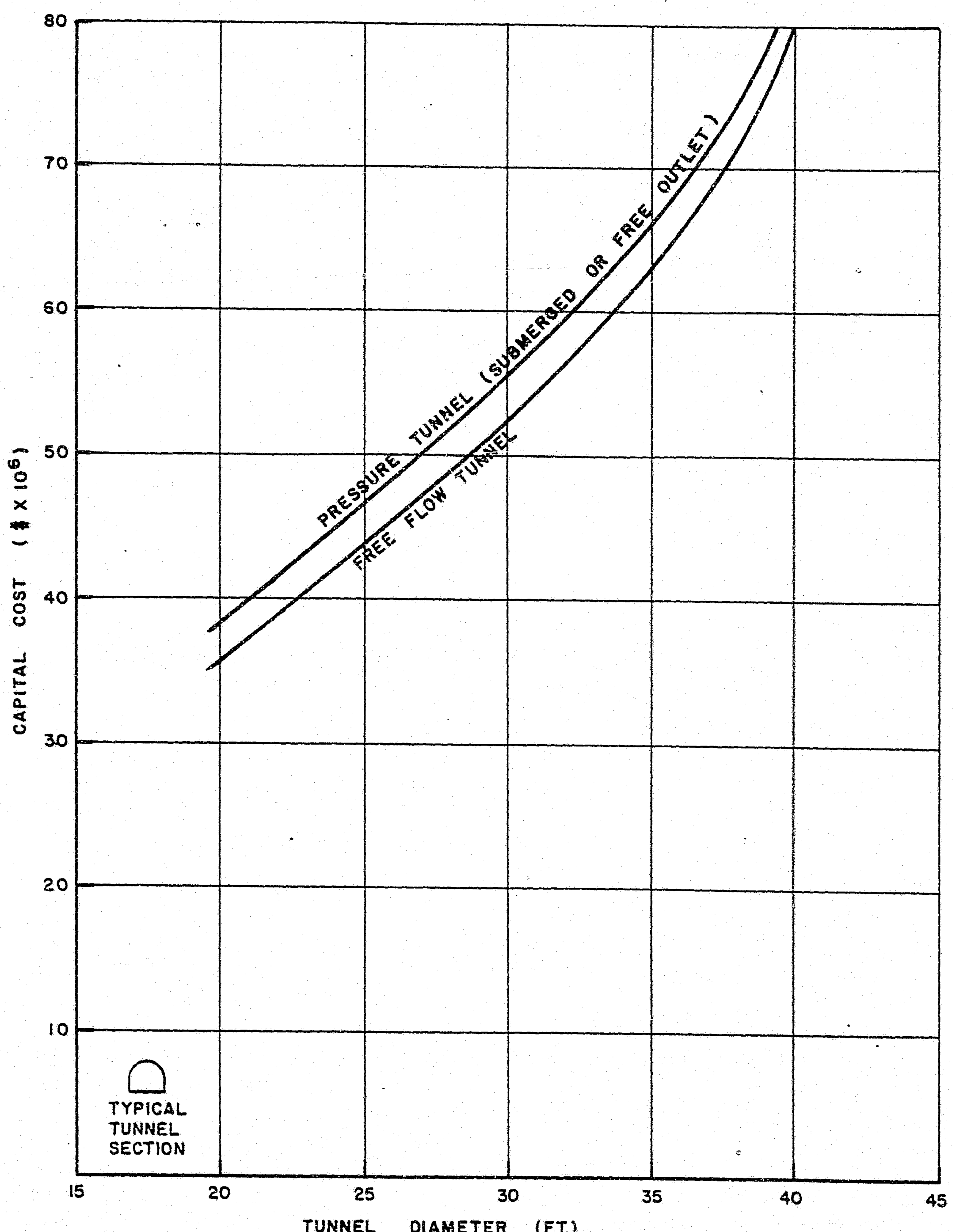
FIGURE B.22



WATANA DIVERSION
UPSTREAM COFFERDAM COSTS

ACRES

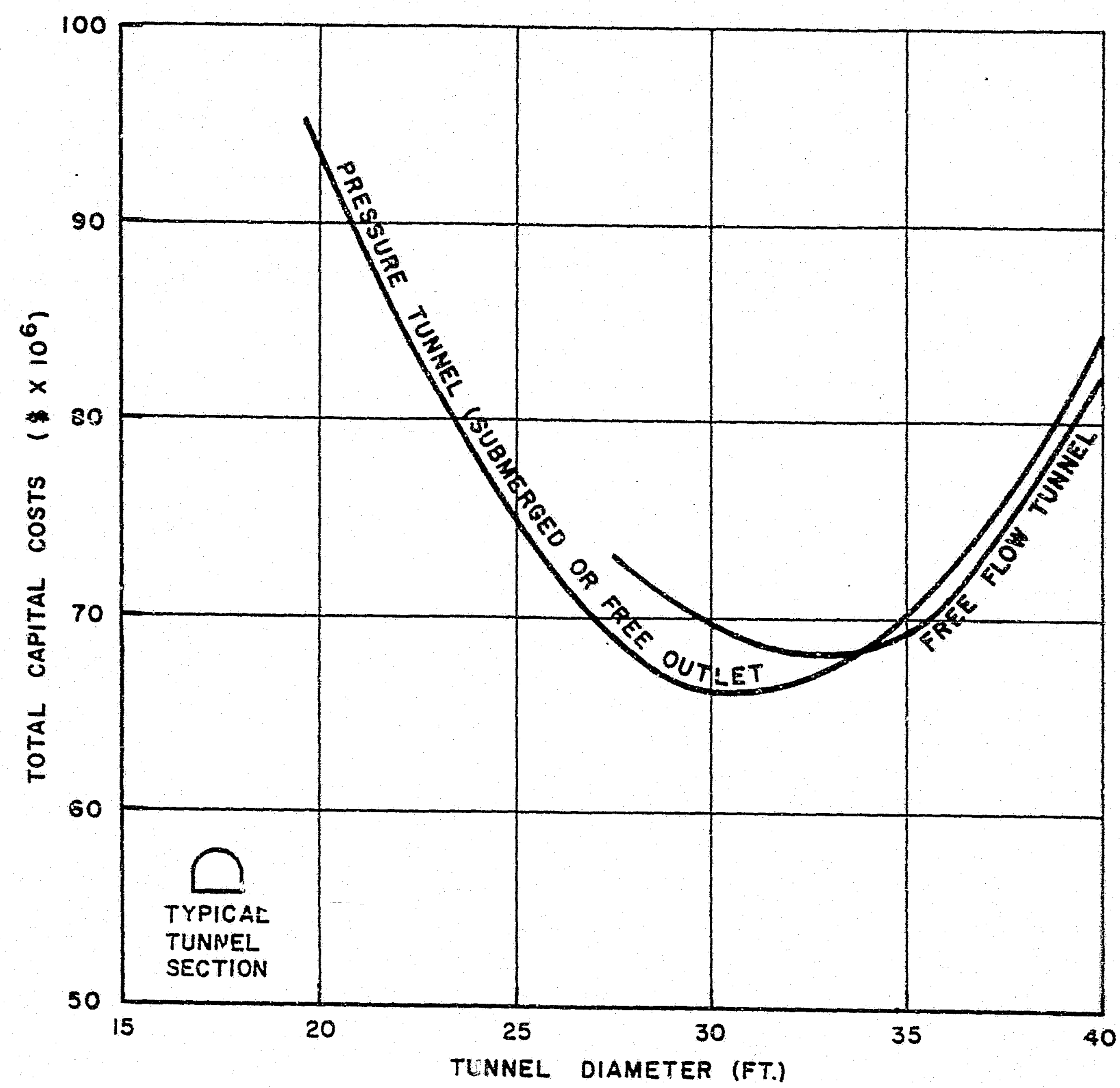
FIGURE B.23



WATANA DIVERSION
TUNNEL COST / TUNNEL DIAMETER

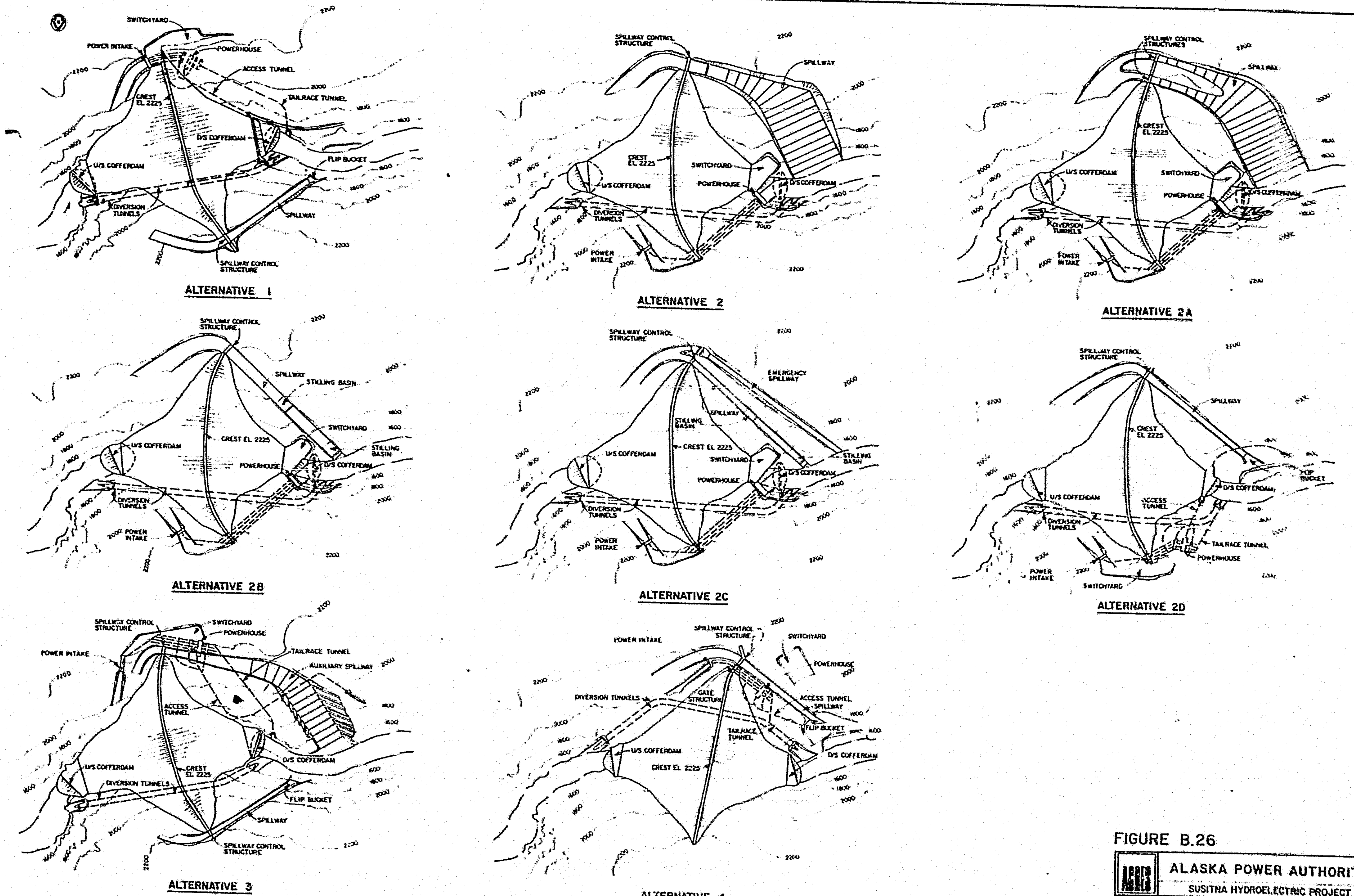
ACRES

FIGURE B.24



WATANA DIVERSION
TOTAL COST / TUNNEL DIAMETER

FIGURE B.25



WATANA
PRELIMINARY SCHEMES

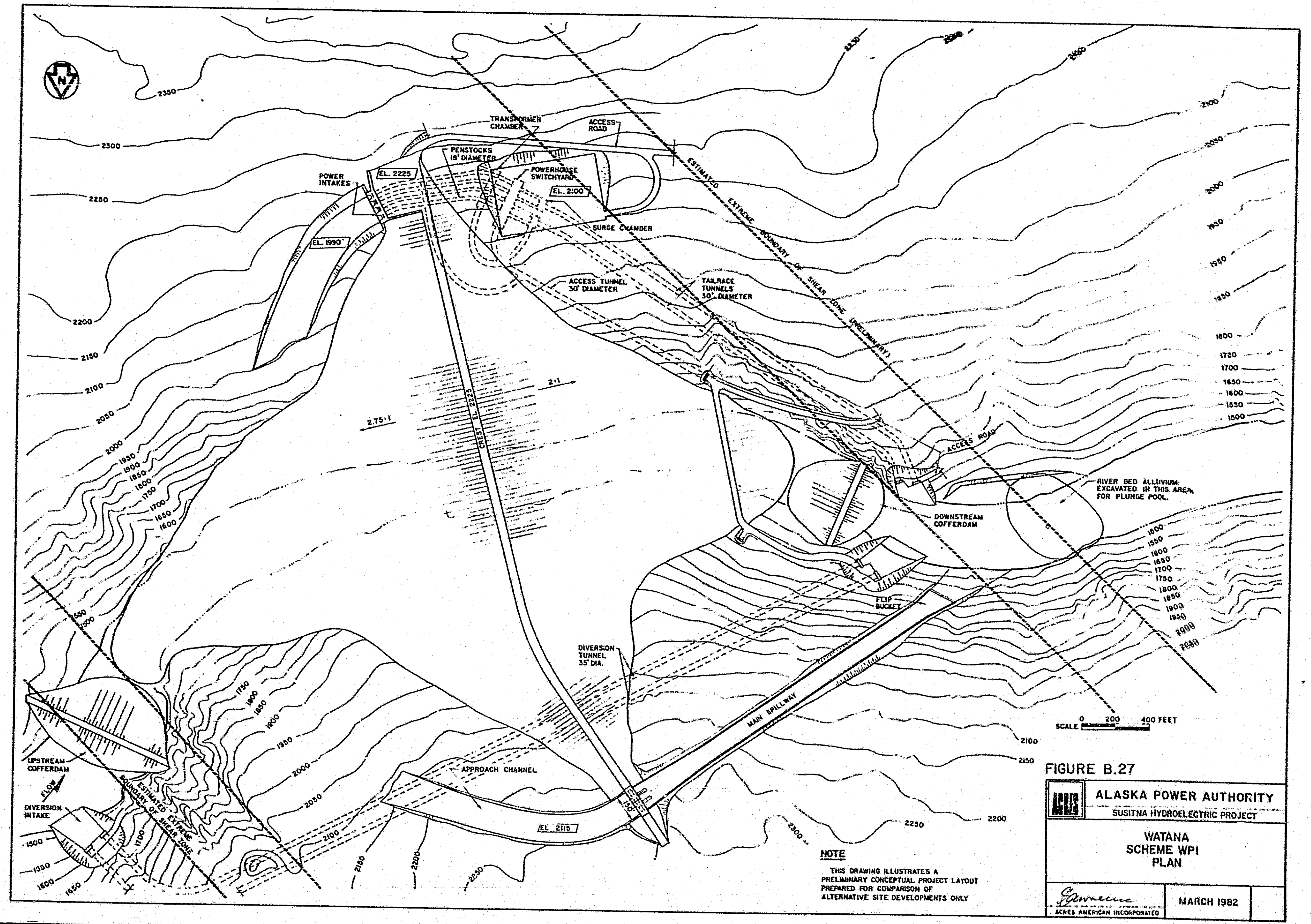
FIGURE B.26

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

WATANA
PRELIMINARY SCHEMES

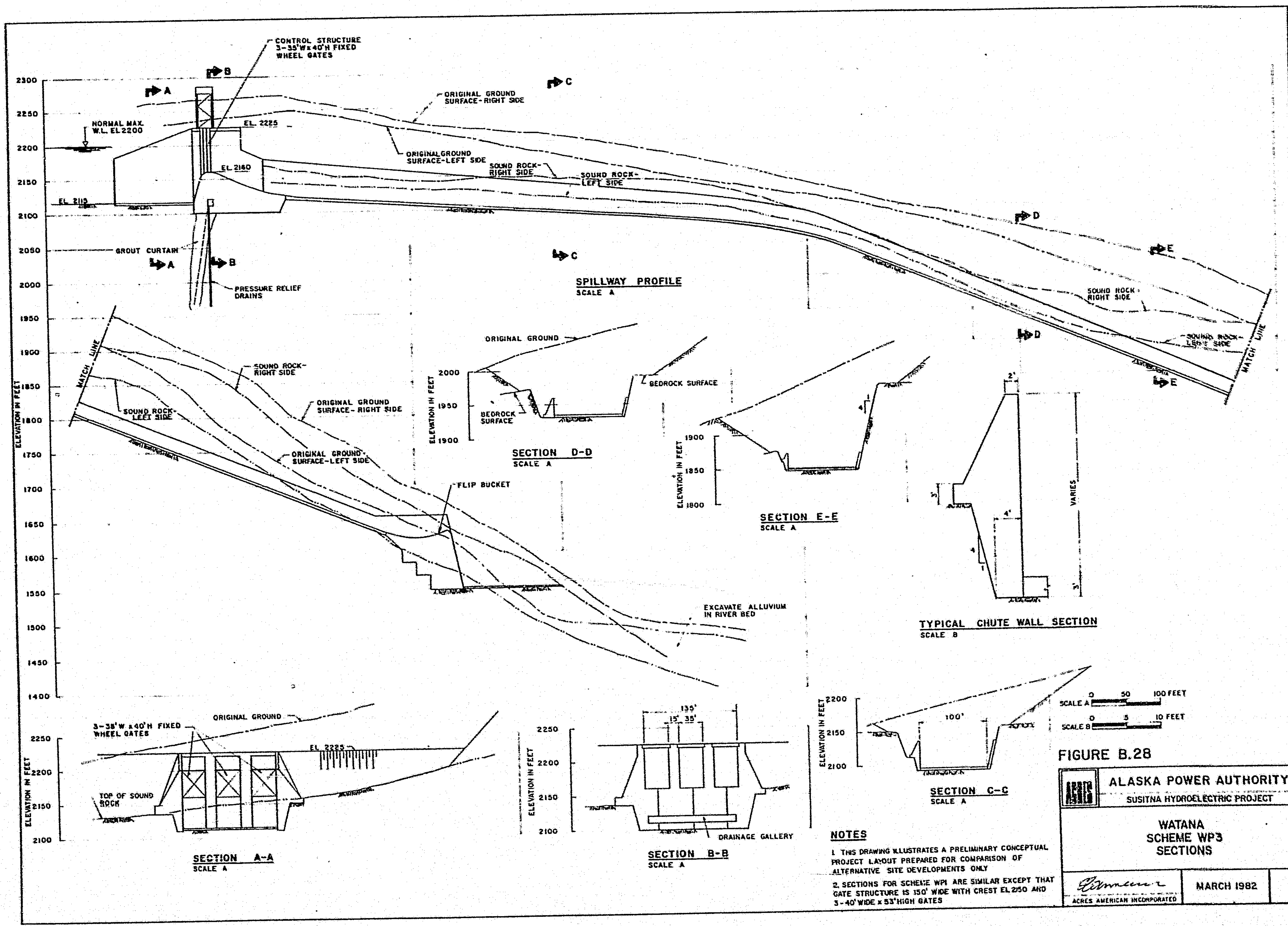
ACRES AMERICAN INCORPORATED

MARCH 1982



TRANSFORMER CHAMBER
ACCESS ROAD
PENSTOCKS 18' DIAMETER
POWERHOUSE SWITCHYARD
POWER INTAKES
EL. 2225
EL. 2100
EL. 1990
SURGE CHAMBER
ACCESS TUNNEL 30' DIAMETER
TAILRACE TUNNELS 30' DIAMETER
ESTIMATED EXTREME BOUNDARY OF SHEAR ZONE (PRELIMINARY)
CREST EL. 2225
2.75:1
2:1
ACCESS ROAD
RIVER BED ALLUVIUM EXCAVATED IN THIS AREA FOR PLUNGE POOL.
DOWNSTREAM COFFERDAM
FLIP BUCKET
DIVERSION TUNNEL 35' DIA.
MAIN SPILLWAY
APPROACH CHANNEL
EL. 2115
UPSTREAM COFFERDAM
FLOW
DIVERSION INTAKE
ESTIMATED EXTREME BOUNDARY OF SHEAR ZONE
SCALE 0 200 400 FEET
NOTE
THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA SCHEME WP1 PLAN
ACRES AMERICAN INCORPORATED
MARCH 1982

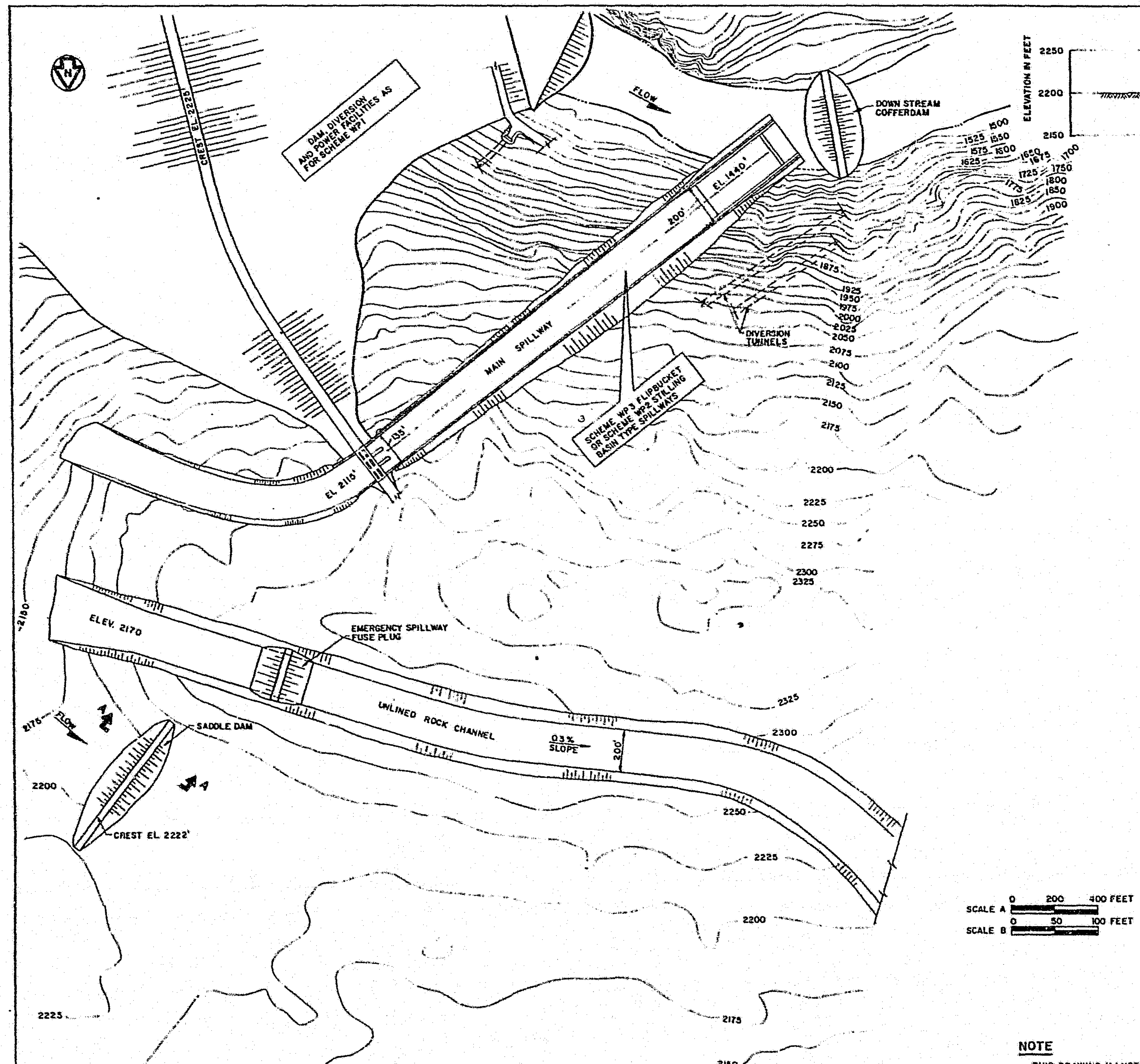
FIGURE B.27



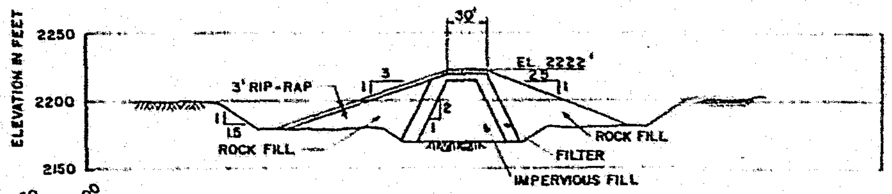
NOTES

1. THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY
2. SECTIONS FOR SCHEME WP1 ARE SIMILAR EXCEPT THAT GATE STRUCTURE IS 150' WIDE WITH CREST EL 2150 AND 3 - 40' WIDE x 53' HIGH GATES

FIGURE B.28



GENERAL ARRANGEMENT
SCALE: A



SECTION A-A
SCALE: B

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.29

ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

WATANA
SCHEME WP 2 & WP 3
PLAN & SECTIONS

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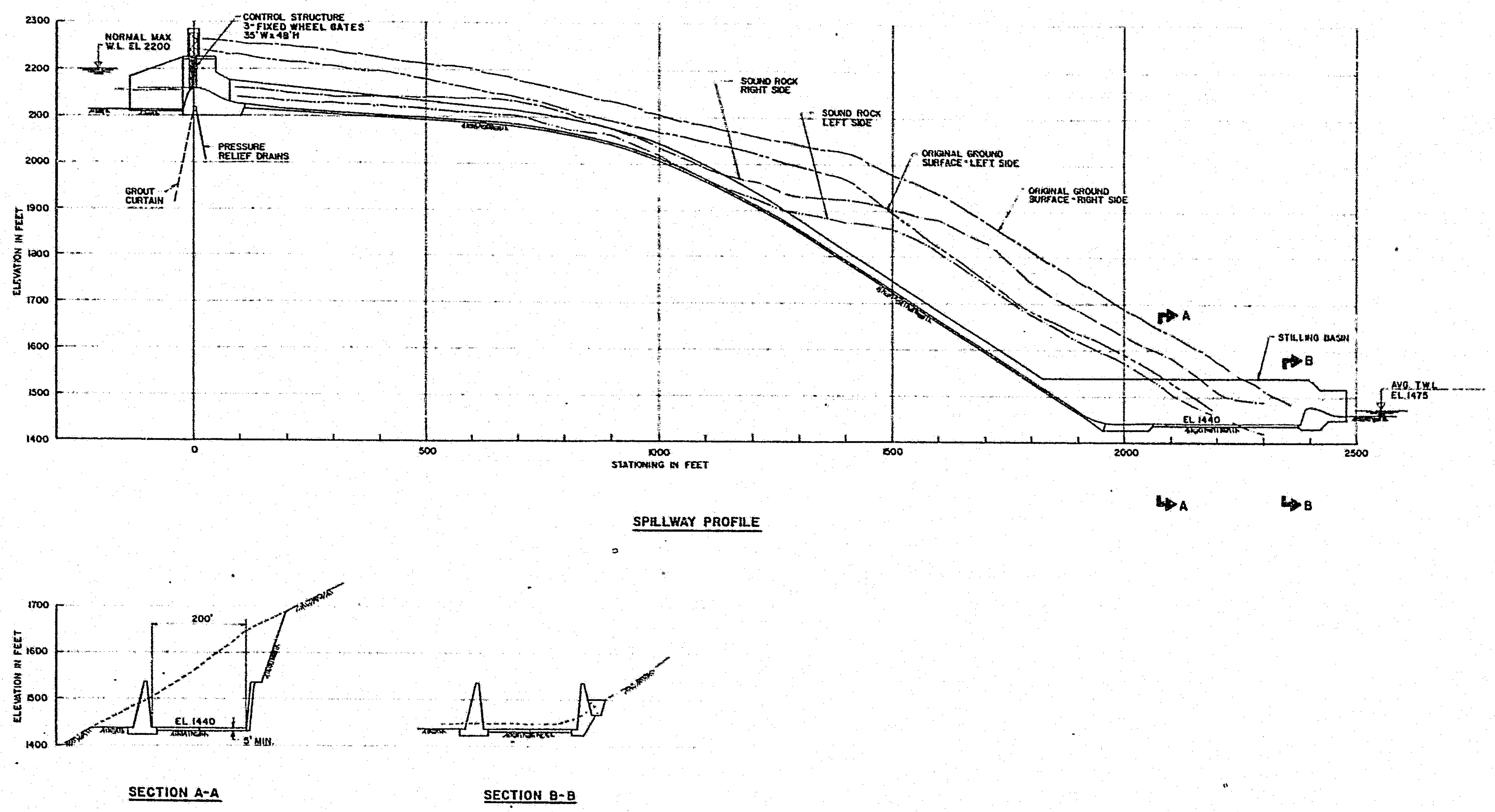
MARCH 1982



NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.30

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

WATANA
SCHEME WP2
SECTIONS

ACRES AMERICAN INCORPORATED

MARCH 1982

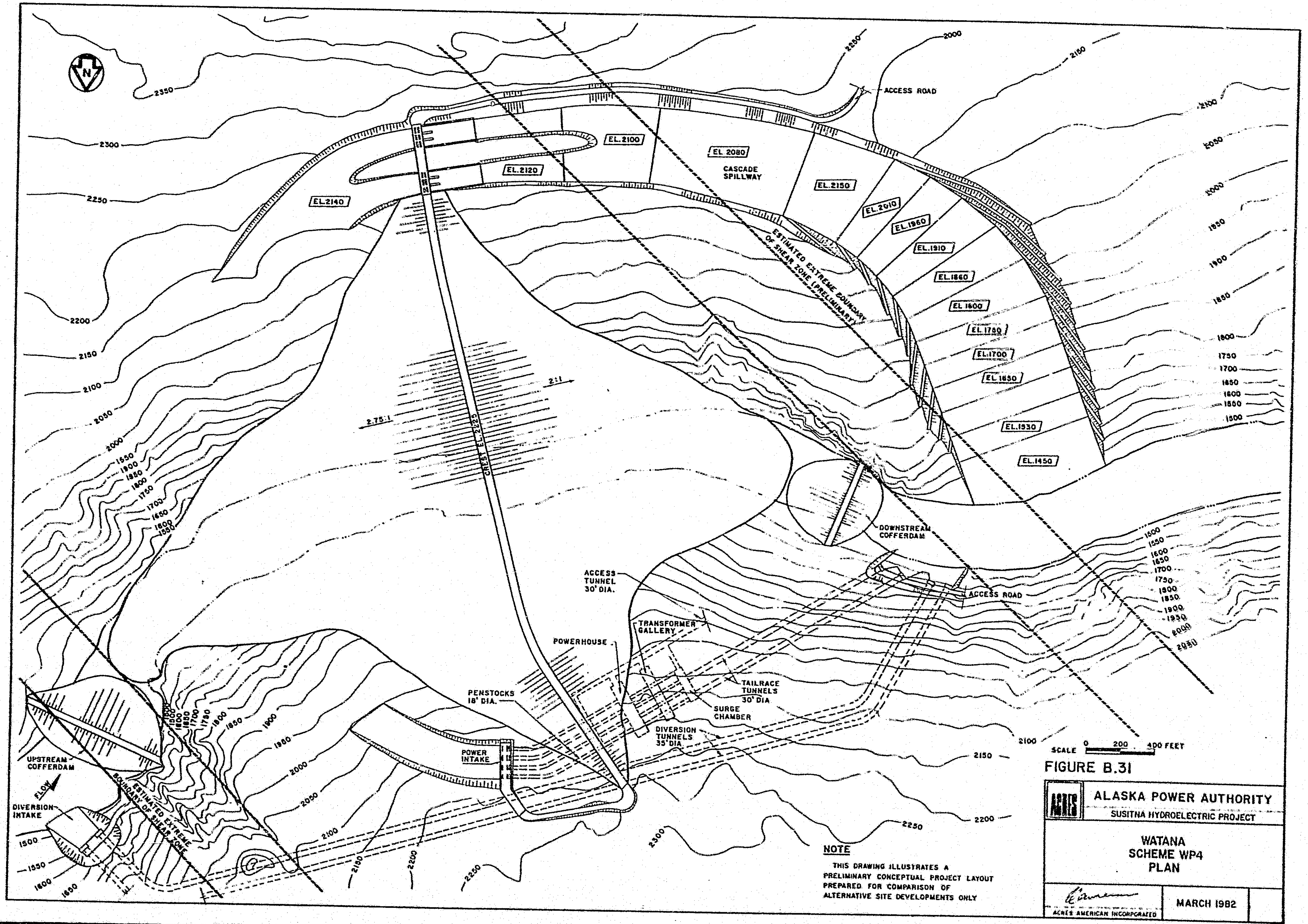


FIGURE B.31

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

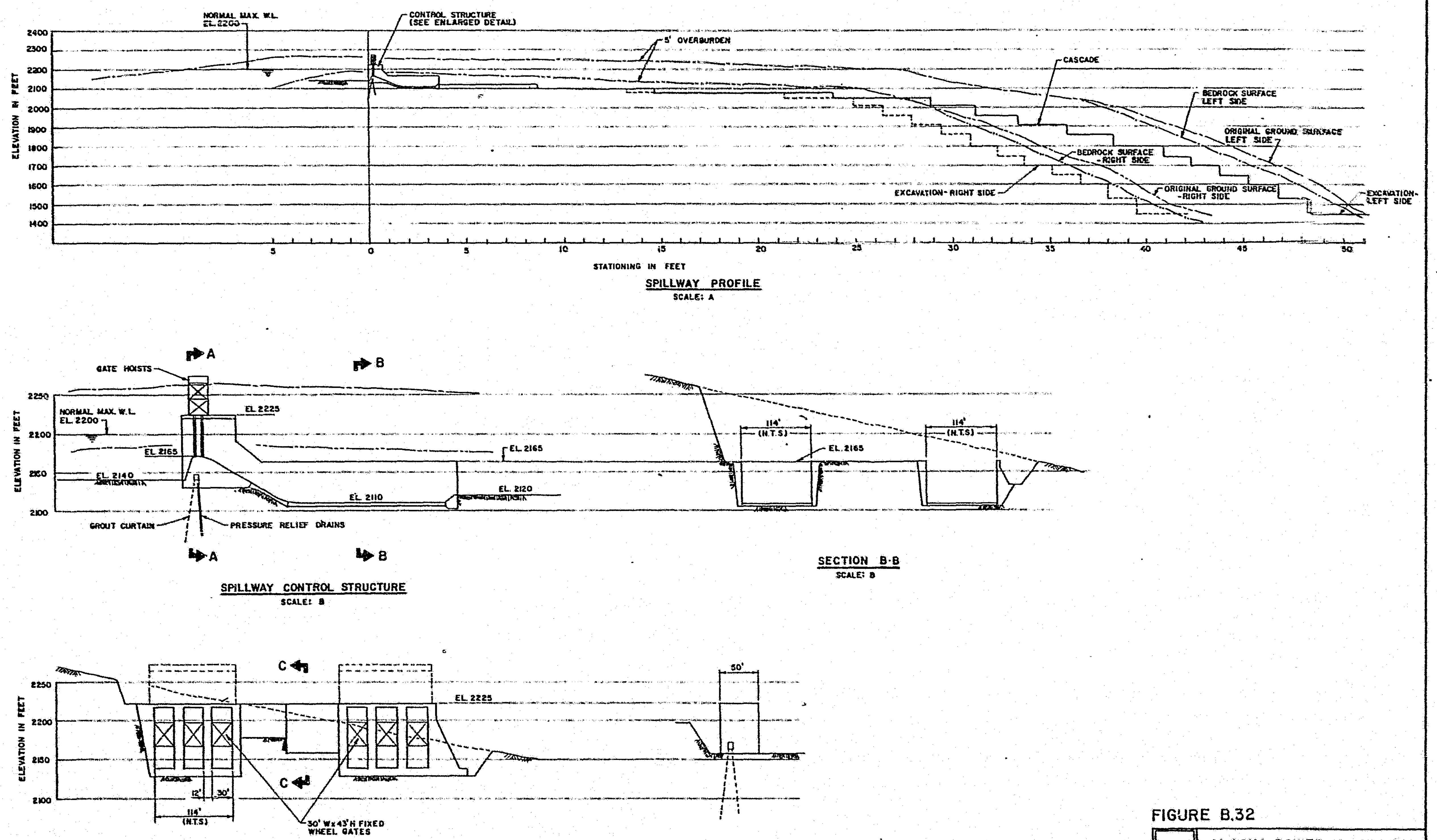
WATANA
SCHEME WP4
PLAN

ACRES AMERICAN INCORPORATED

MARCH 1982

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY



SCALE A 0 200 400 FEET

SCALE B 0 50 100 FEET

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.32

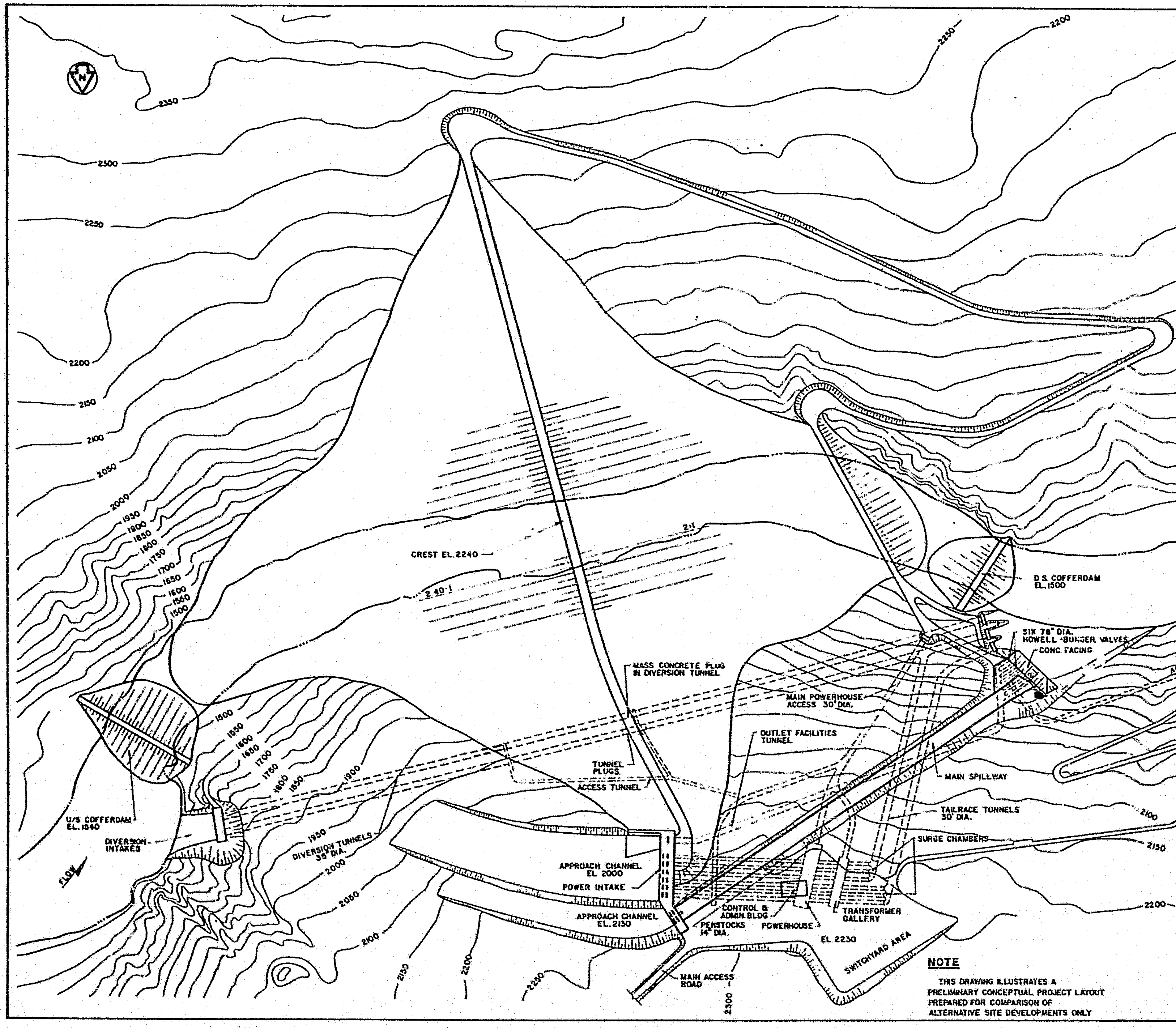
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

WATANA
SCHEME WP4
SECTIONS

ACRES AMERICAN INCORPORATED

MARCH 1982



NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.33

ACRES

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

WATANA
SCHEME WP3A

ACRES AMERICAN INCORPORATED

MARCH 1982

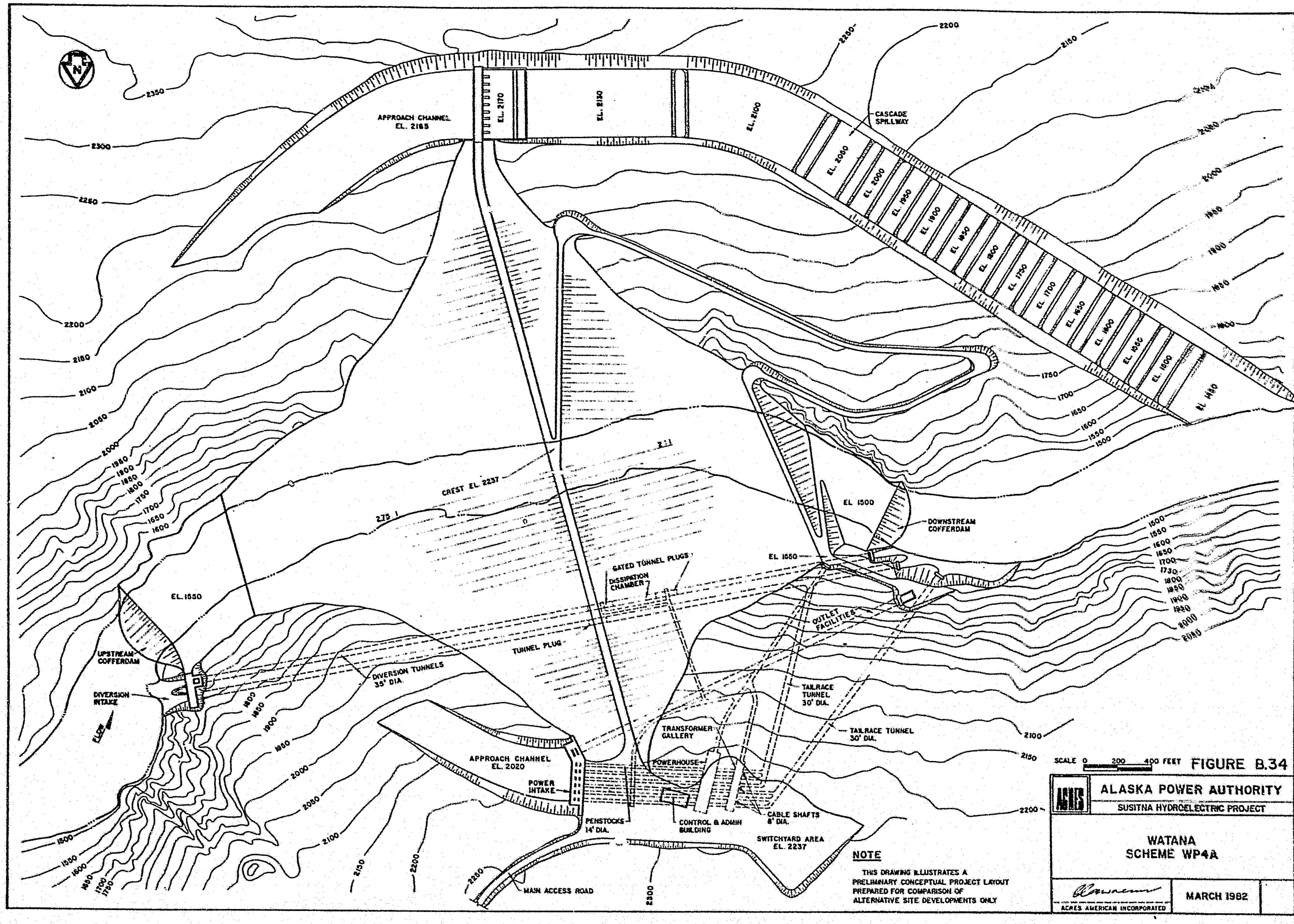
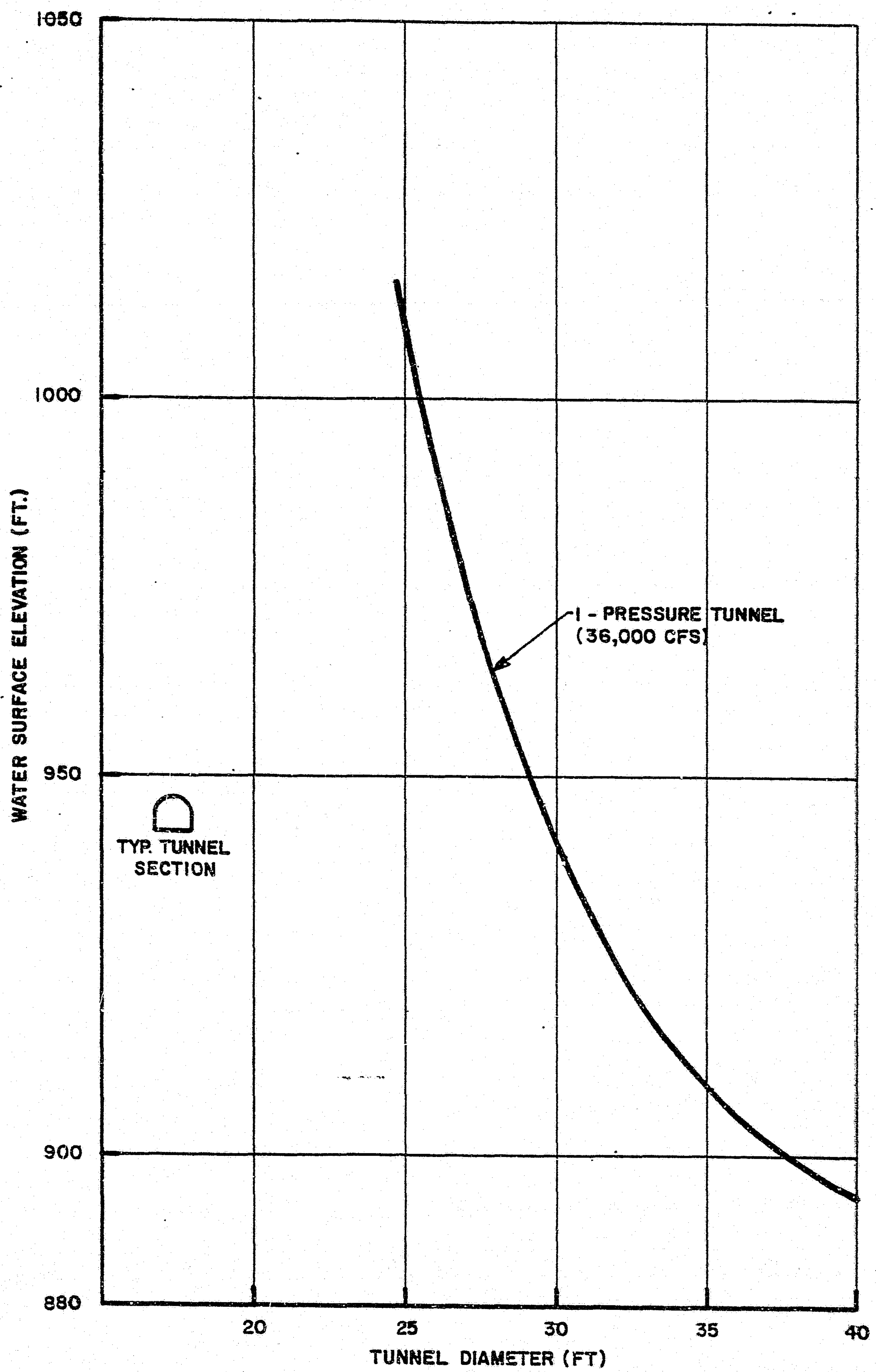

APPROACH CHANNEL
EL. 2165
EL. 2170
EL. 2150
EL. 2100
CASCADE
SPILLWAY
EL. 2050
EL. 2000
EL. 1950
EL. 1900
EL. 1850
EL. 1800
EL. 1750
EL. 1700
EL. 1650
EL. 1600
EL. 1550
EL. 1500
EL. 1480
CREST EL. 2237
2.75 : 1
2 : 1
EL. 1500
DOWNSTREAM
COFFERDAM
EL. 1550
GATED TUNNEL PLUGS
DISSIPATION
CHAMBER
EL. 1550
OUTLET
FACILITIES
TUNNEL PLUG
UPSTREAM
COFFERDAM
DIVERSION TUNNELS
35' DIA.
DIVERSION
INTAKE
FLOW
TAILRACE
TUNNEL
30' DIA.
TAILRACE TUNNEL
30' DIA.
TRANSFORMER
GALLERY
POWERHOUSE
APPROACH CHANNEL
EL. 2020
POWER
INTAKE
PENSTOCKS
14' DIA.
CONTROL & ADMIN
BUILDING
CABLE SHAFTS
8' DIA.
SWITCHYARD AREA
EL. 2237
MAIN ACCESS ROAD
SCALE 0 200 400 FEET
NOTE
THIS DRAWING ILLUSTRATES A
PRELIMINARY CONCEPTUAL PROJECT LAYOUT
PREPARED FOR COMPARISON OF
ALTERNATIVE SITE DEVELOPMENTS ONLY
ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT
WATANA
SCHEME WP4A
ACRES AMERICAN INCORPORATED
MARCH 1982

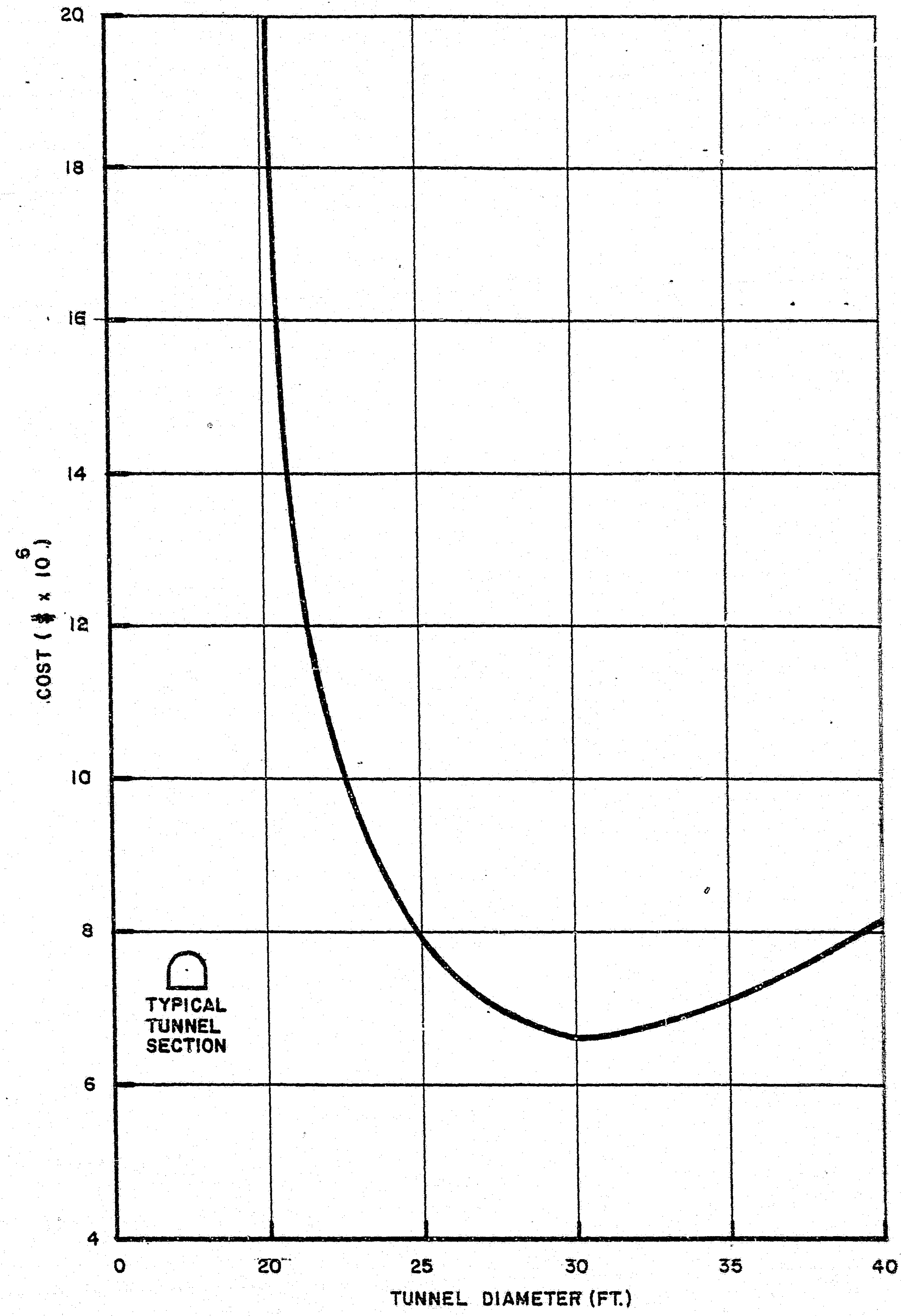
FIGURE B.34



DEVIL CANYON DIVERSION
HEADWATER ELEVATION / TUNNEL DIAMETER

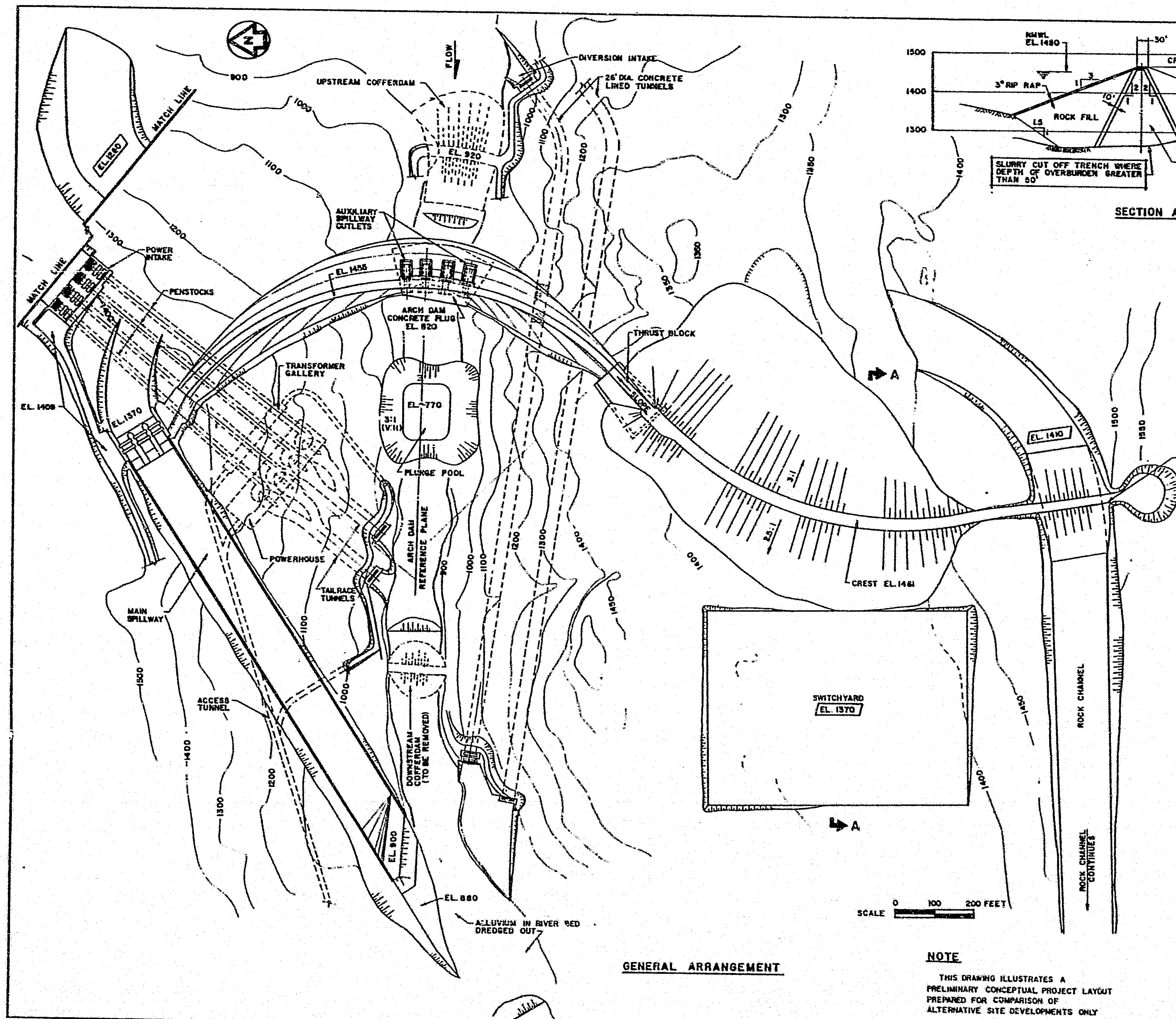
FIGURE B.35

ACRES



DEVIL CANYON DIVERSION
TOTAL COST / TUNNEL DIAMETER

FIGURE B.36



NOTE

POWERHOUSE LOCATION SUBJECT TO OPTIMIZATION STUDIES OF DOWNSTREAM LOCATIONS.

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.37

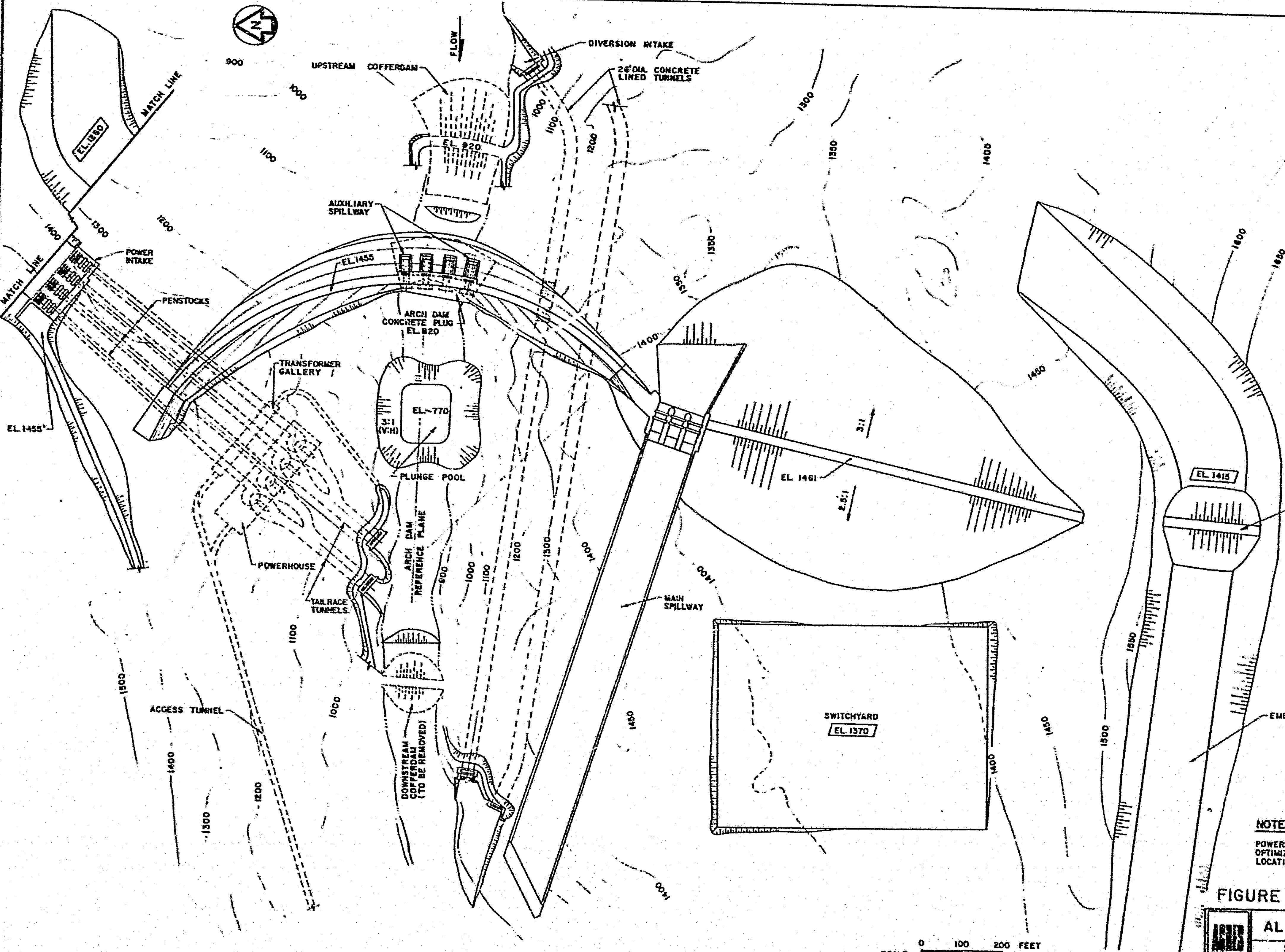
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DEVIL CANYON SCHEME DC1

ACRES AMERICAN INCORPORATED

MARCH 1982



GENERAL ARRANGEMENT

NOTE

POWERHOUSE LOCATION SUBJECT TO OPTIMIZATION STUDIES OF DOWNSTREAM LOCATIONS.

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

FIGURE B.38

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DEVIL CANYON SCHEME DC2

ACRES AMERICAN INCORPORATED

MARCH 1982

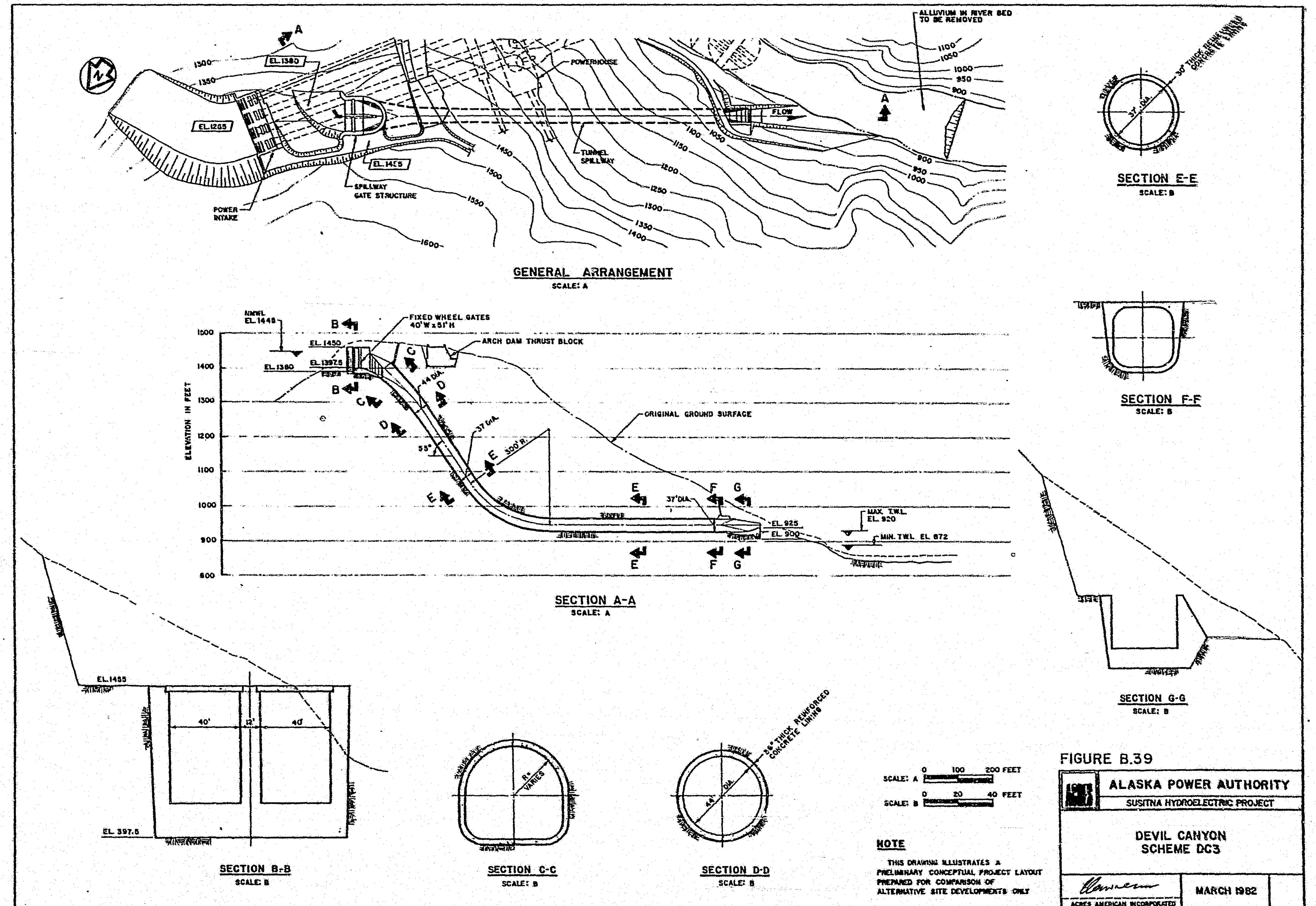


FIGURE B.39

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DEVIL CANYON SCHEME DC3

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

ACRES AMERICAN INCORPORATED

MARCH 1982

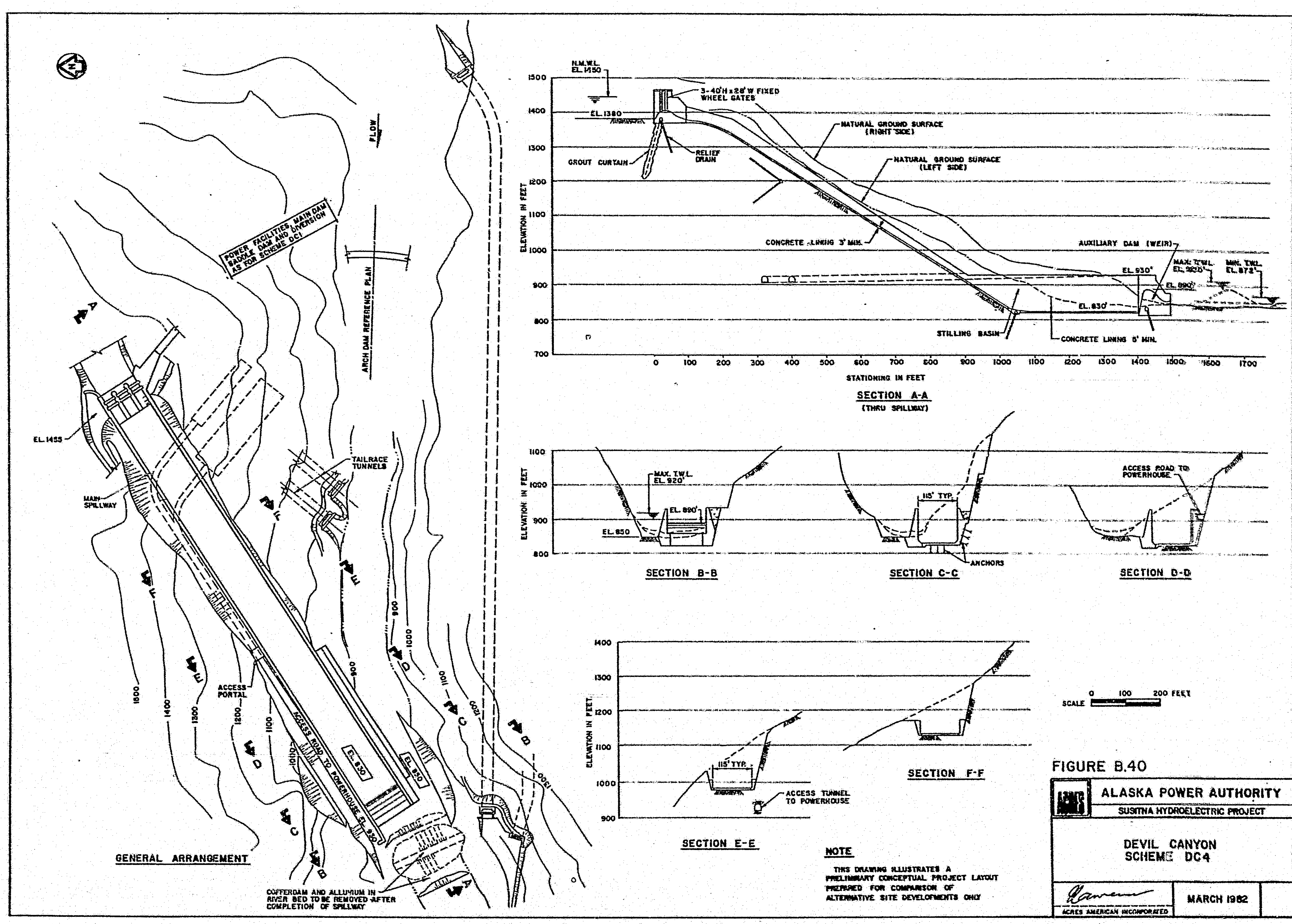


FIGURE B.40

NOTE

THIS DRAWING ILLUSTRATES A PRELIMINARY CONCEPTUAL PROJECT LAYOUT PREPARED FOR COMPARISON OF ALTERNATIVE SITE DEVELOPMENTS ONLY

ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

DEVIL CANYON SCHEME DC4

ACRES AMERICAN INCORPORATED

MARCH 1982

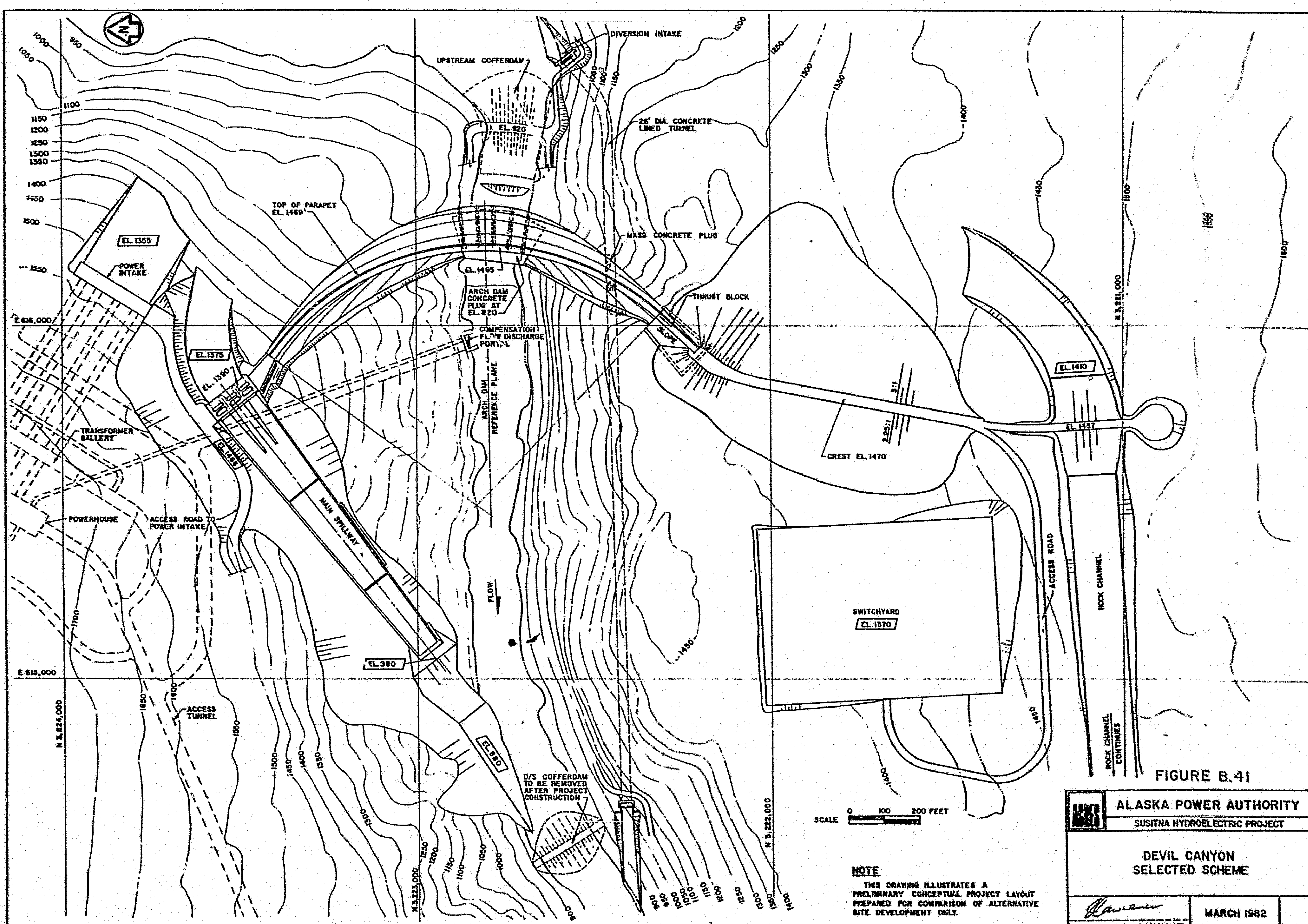


FIGURE B.41

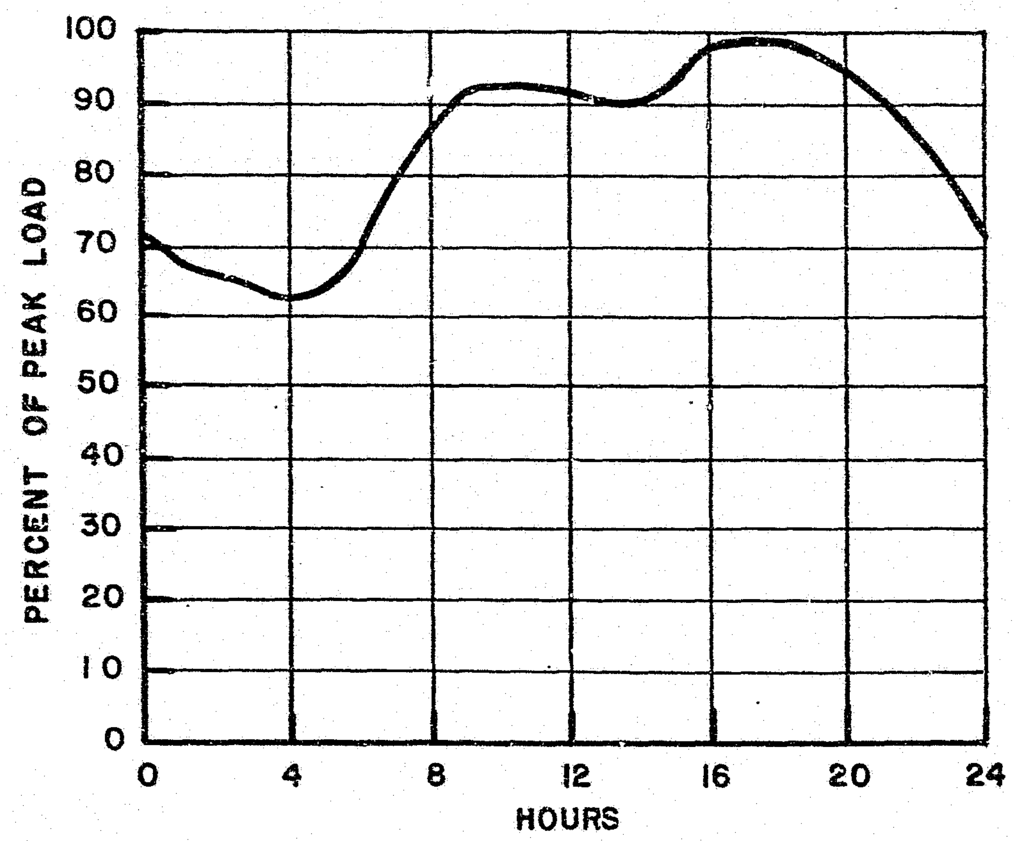
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

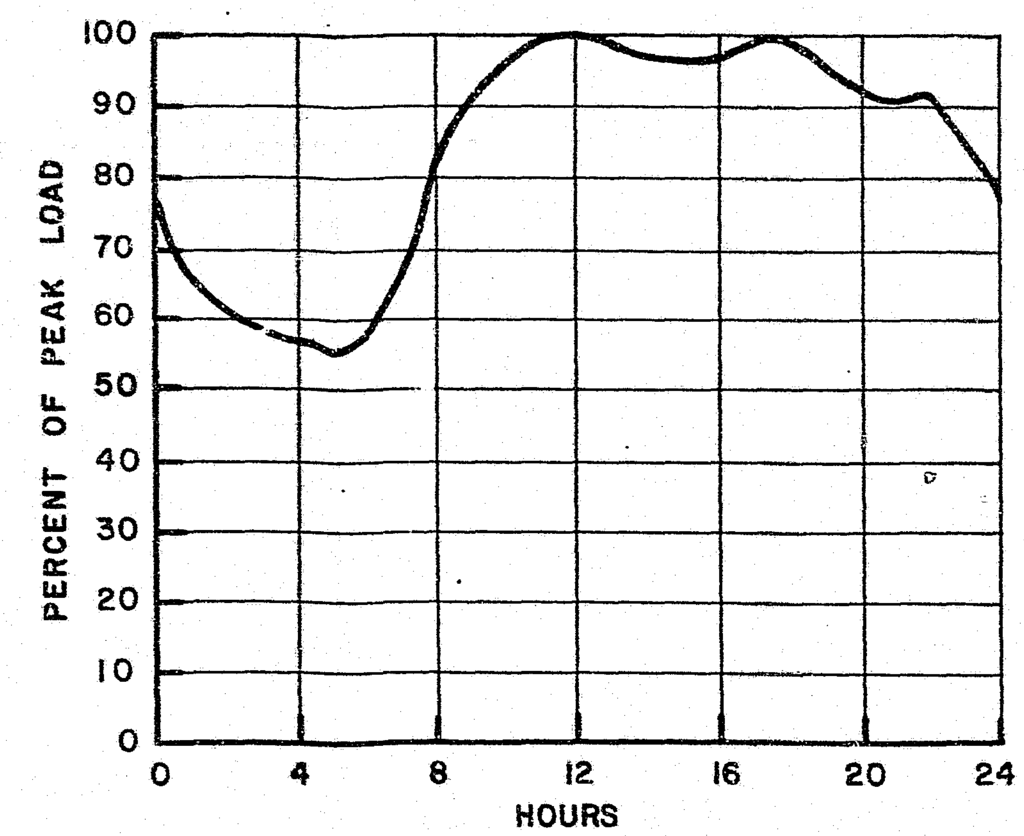
DEVIL CANYON
SELECTED SCHEME

ACRES AMERICAN INCORPORATED

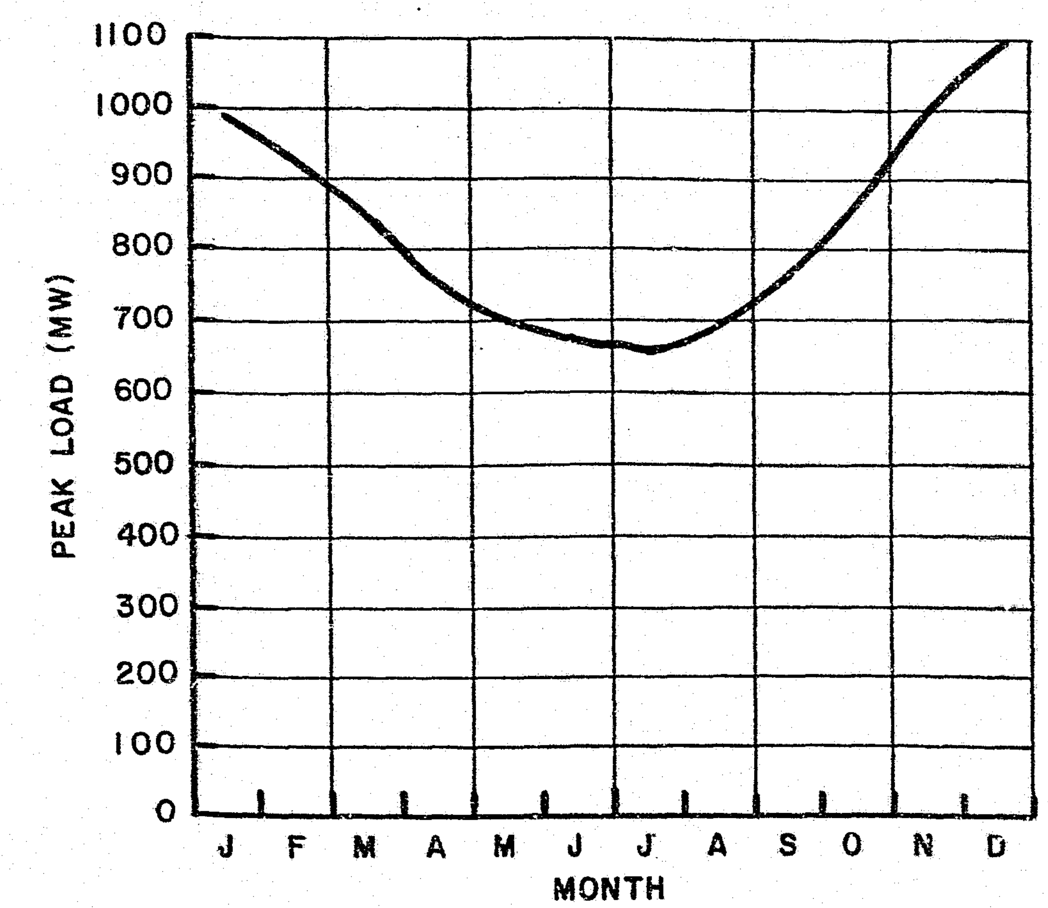
MARCH 1982



WINTER WEEKDAY
HOURLY LOAD VARIATION



SUMMER WEEKDAY
HOURLY LOAD VARIATION



LOAD VARIATION
IN YEAR 2000

NOTE: PEAK MW DECEMBER 2000 AD = 1084 MW

NOTE: PEAK MW JULY 2000 AD = 658 MW

TYPICAL LOAD VARIATION
IN ALASKA RAILBELT SYSTEM

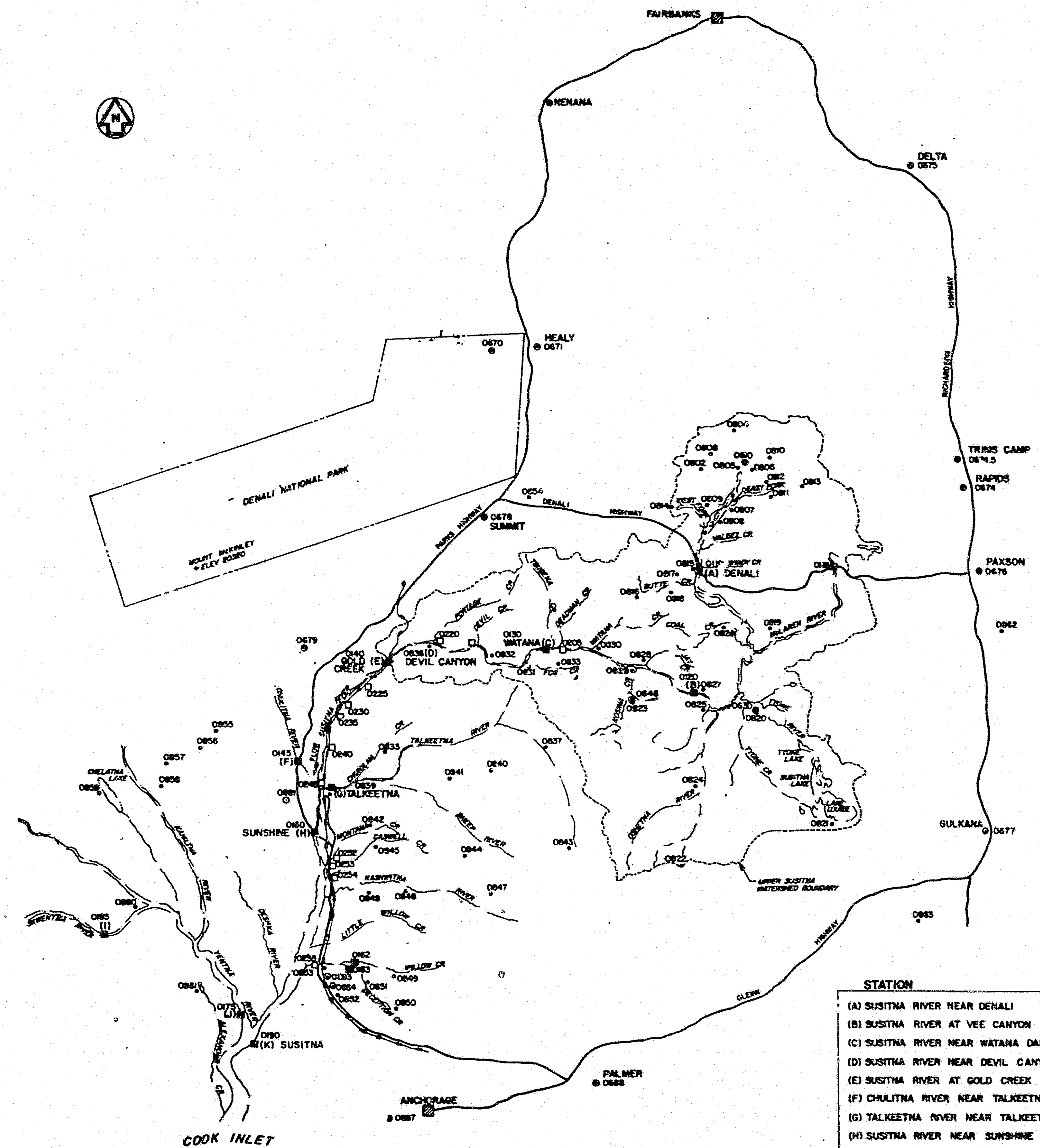
FIGURE B.42

DATA COLLECTED

		INDEX NUMBERING
■	STREAMFLOW - CONTINUOUS RECORD	0100
□	STREAMFLOW - PARTIAL RECORD	0200
⊕	WATER QUALITY	0300
T	WATER TEMPERATURE	0400
▲	SEDIMENT DISCHARGE	0500
⊙	CLIMATE	0600
⊢	FREEZING RAIN AND INCLOUD ICING	0700
•	SNOW COURSE	0800
↖	SNOW CREEP	0900

NOTES

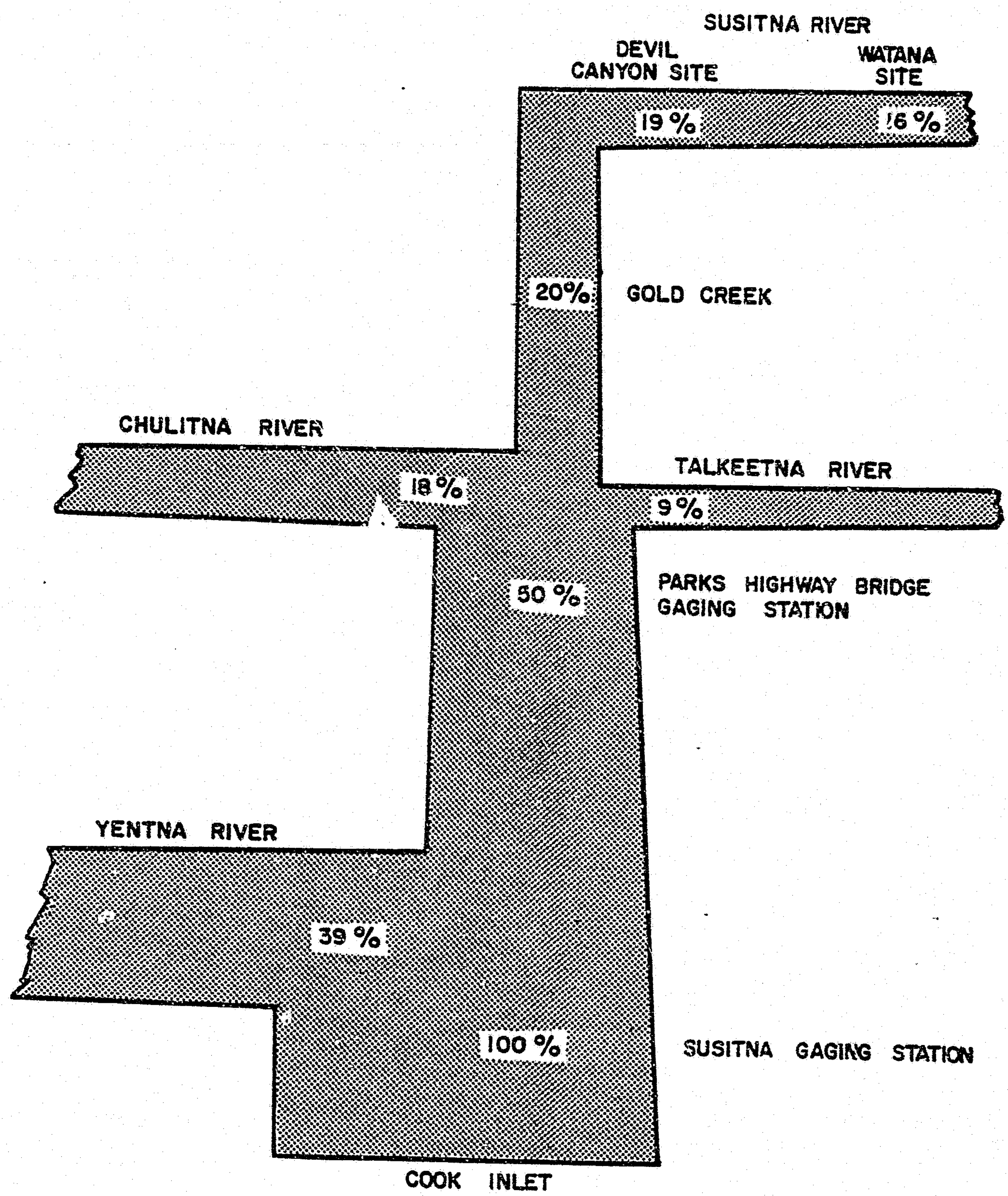
1. PARAMETERS MEASURED LISTED IN APPENDIX B1
2. CONTINUOUS WATER QUALITY MONITOR INSTALLED
3. DATA COLLECTION 1981 SEASON
4. THE LETTER BEFORE EACH STATION NAME IN THE TABLE IS USED ON THE MAP TO MARK THE APPROXIMATE LOCATION OF THE STATIONS.



STATION	STREAMFLOW GAGING	CREST STAGE GAGE	STAFF GAGE	WATER QUALITY (1)	WATER TEMPERATURE	SEDIMENT DISCHARGE	BEDLOAD SAMPLING (3)	CLIMATE	IN-CLOUD ICING	FREEZING RAIN	SNOW CREEP	PERIOD OF RECORD OF STREAMFLOW
(A) SUSITNA RIVER NEAR DENALI	X		X		X	X		X	X	X		1957-PRESENT
(B) SUSITNA RIVER AT VEE CANYON	X		X	X	X	X						1961-1972 & 1980-PRESENT
(C) SUSITNA RIVER NEAR WATANA DAMSITE	X	X		X[2]	X			X	X	X	X	1980-PRESENT
(D) SUSITNA RIVER NEAR DEVIL CANYON		X	X					X			X	
(E) SUSITNA RIVER AT GOLD CREEK	X			X	X	X	X					1949-PRESENT
(F) CHULITNA RIVER NEAR TALKEETNA	X				X	X	X					1958-1972 & 1980-PRESENT
(G) TALKEETNA RIVER NEAR TALKEETNA	X			X	X	X	X					1964-PRESENT
(H) SUSITNA RIVER NEAR SUNSHINE	X				X	X	X					1981-PRESENT
(I) SKWENTNA RIVER NEAR SKWENTNA	X			X	X	X		X				1959-1980
(J) YENTNA RIVER NEAR SUSITNA STATION	X				X	X						1980-PRESENT
(K) SUSITNA RIVER AT SUSITNA STATION	X			X	X	X						1974-PRESENT

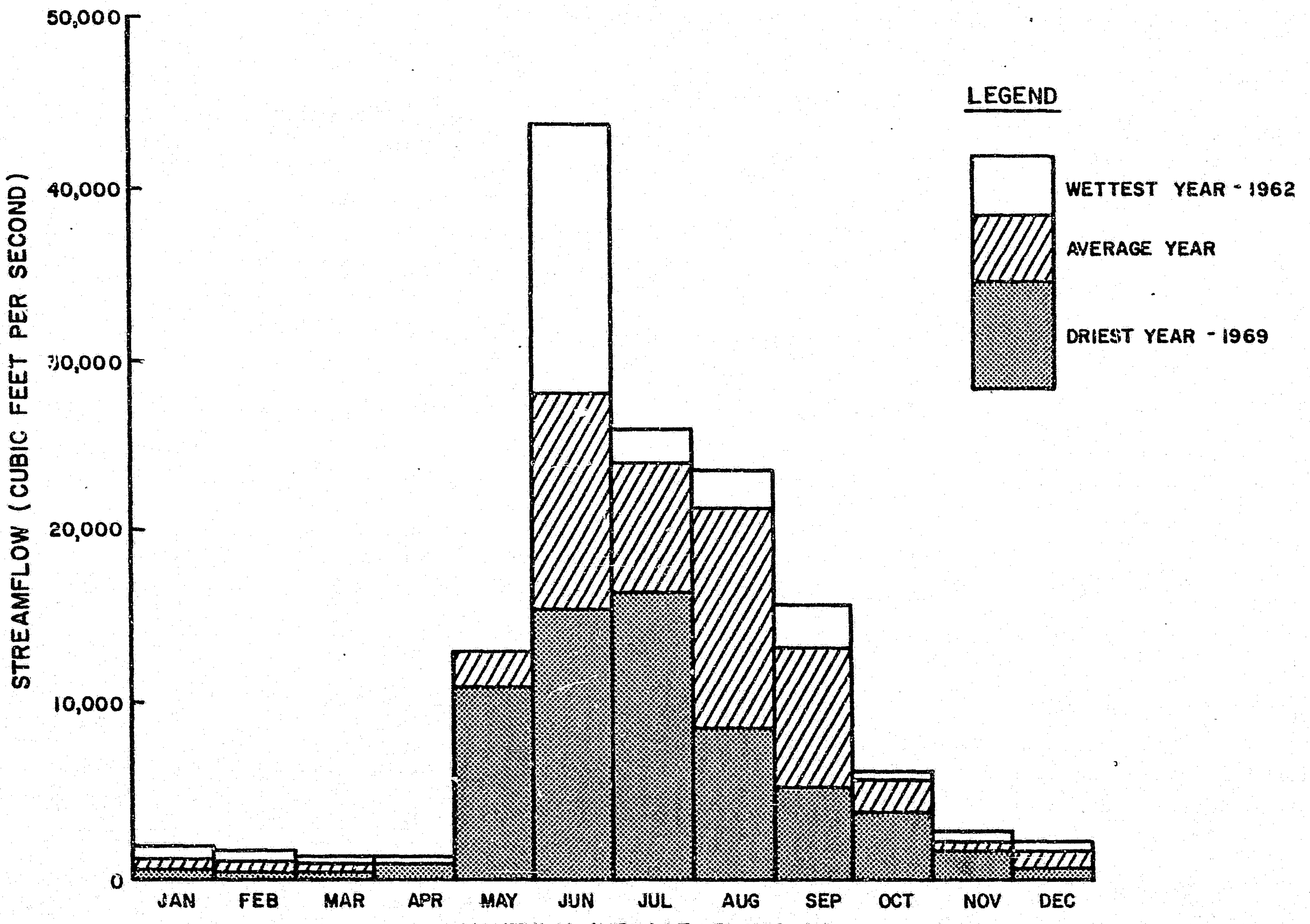
DATA COLLECTION STATIONS
USED IN PMF STUDY

FIGURE B.43



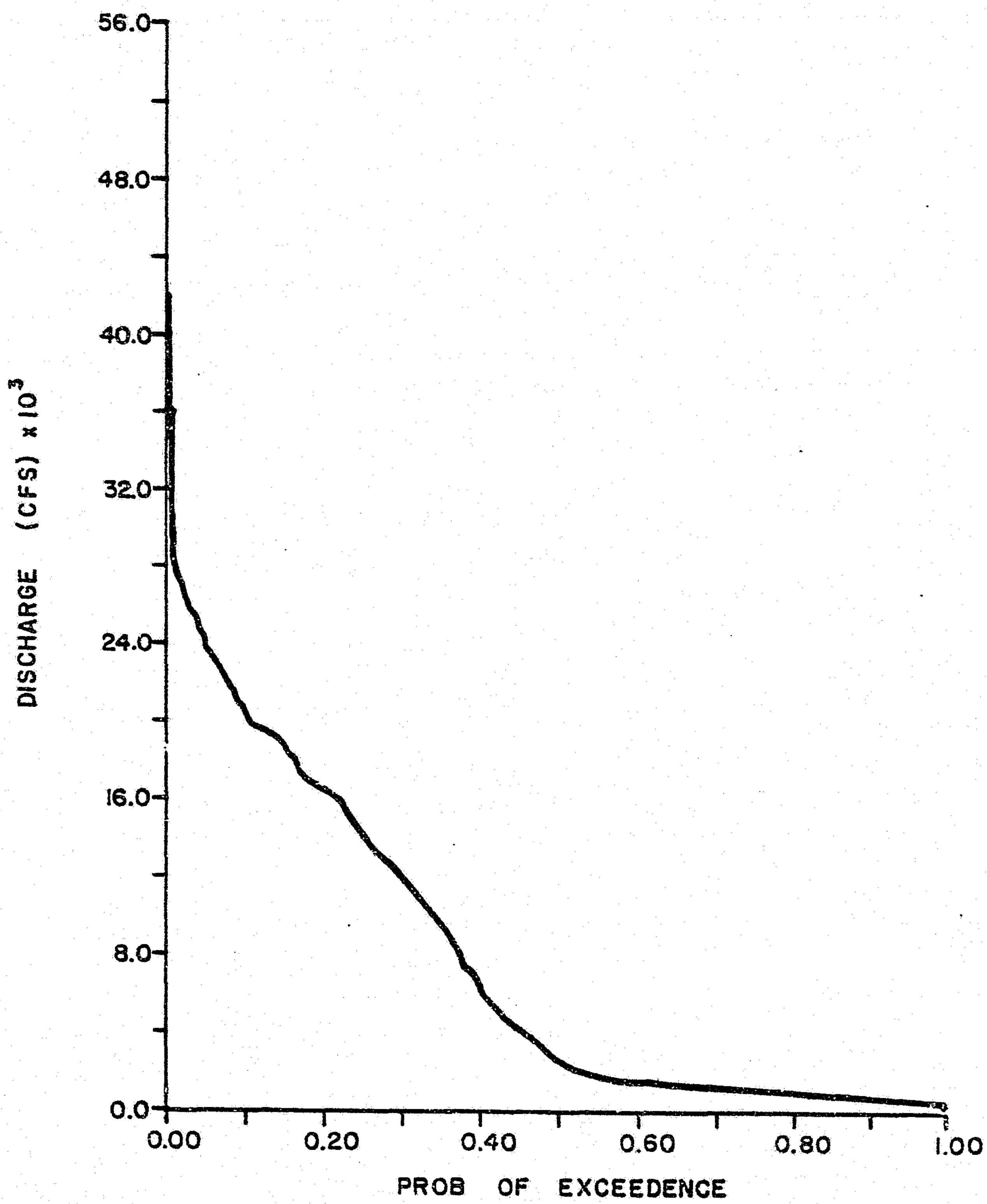
AVERAGE ANNUAL FLOW DISTRIBUTION WITHIN THE SUSITNA RIVER BASIN

FIGURE B.44



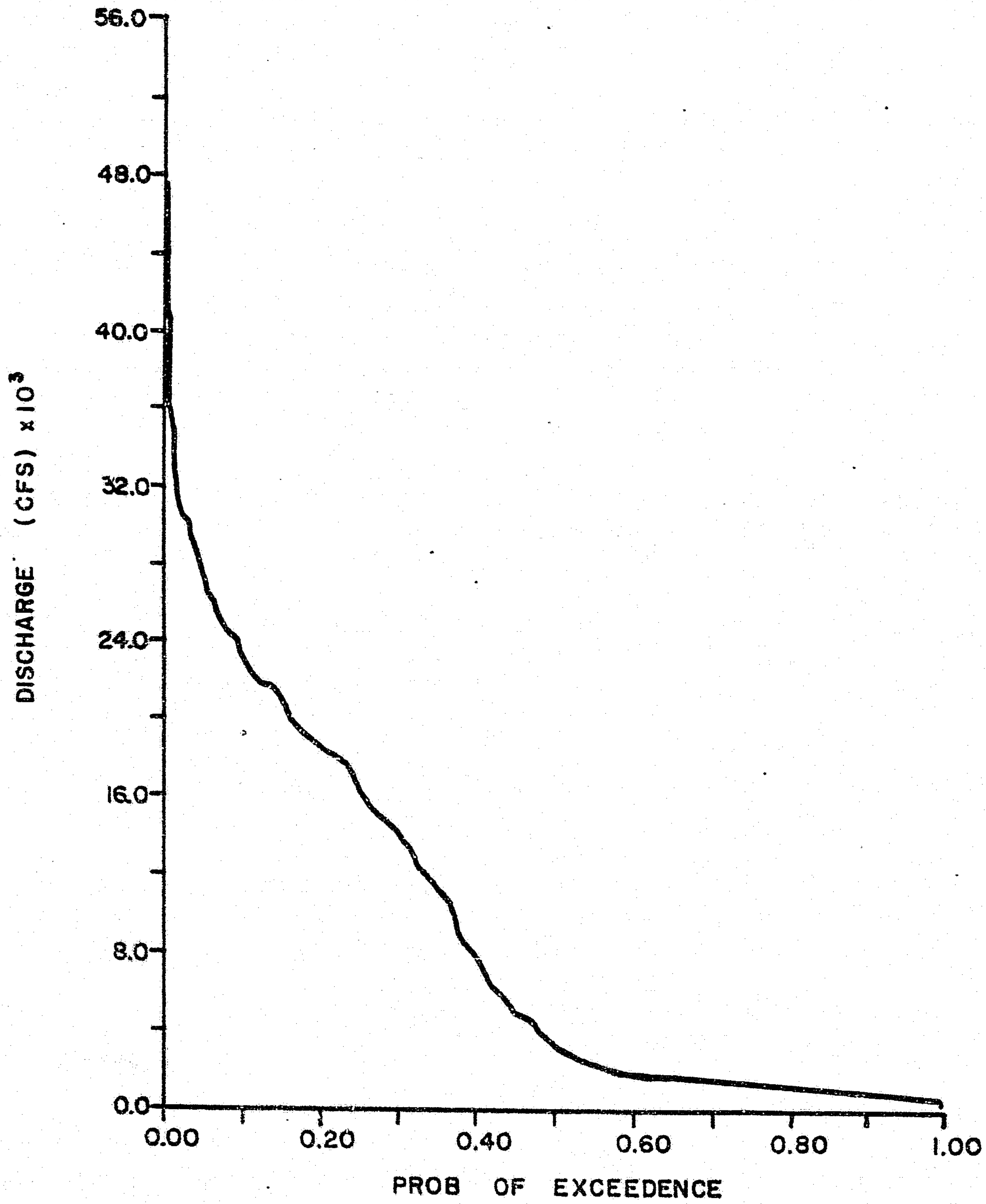
MONTHLY AVERAGE FLOWS IN
THE SUSITNA RIVER AT GOLD CREEK

FIGURE B.45



FLOW DURATION CURVE
MEAN MONTHLY INFLOW
AT WATANA
PRE-PROJECT

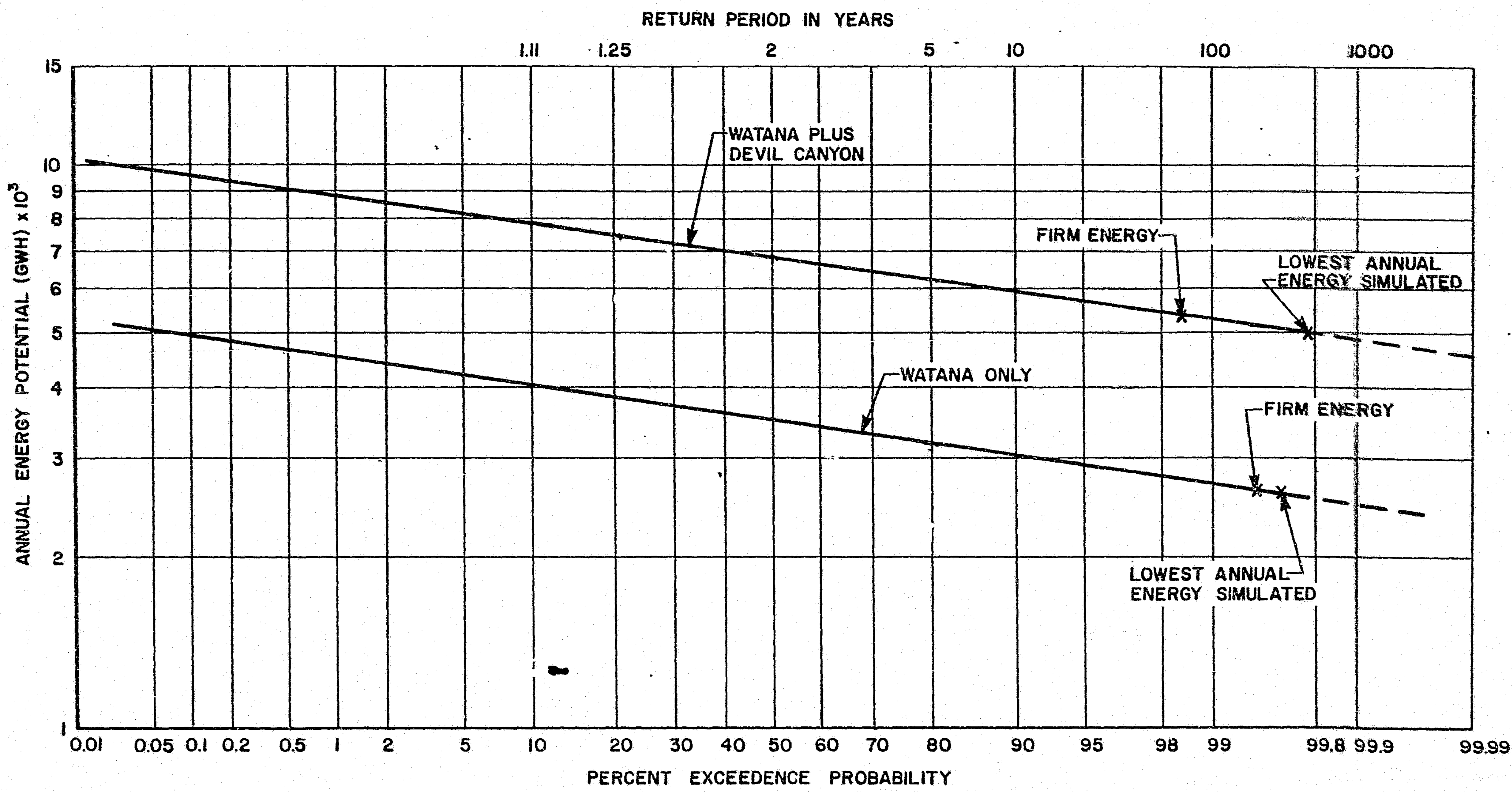
FIGURE B.46



FLOW DURATION CURVE
MEAN MONTHLY INFLOW
AT DEVIL CANYON
PRE-PROJECT

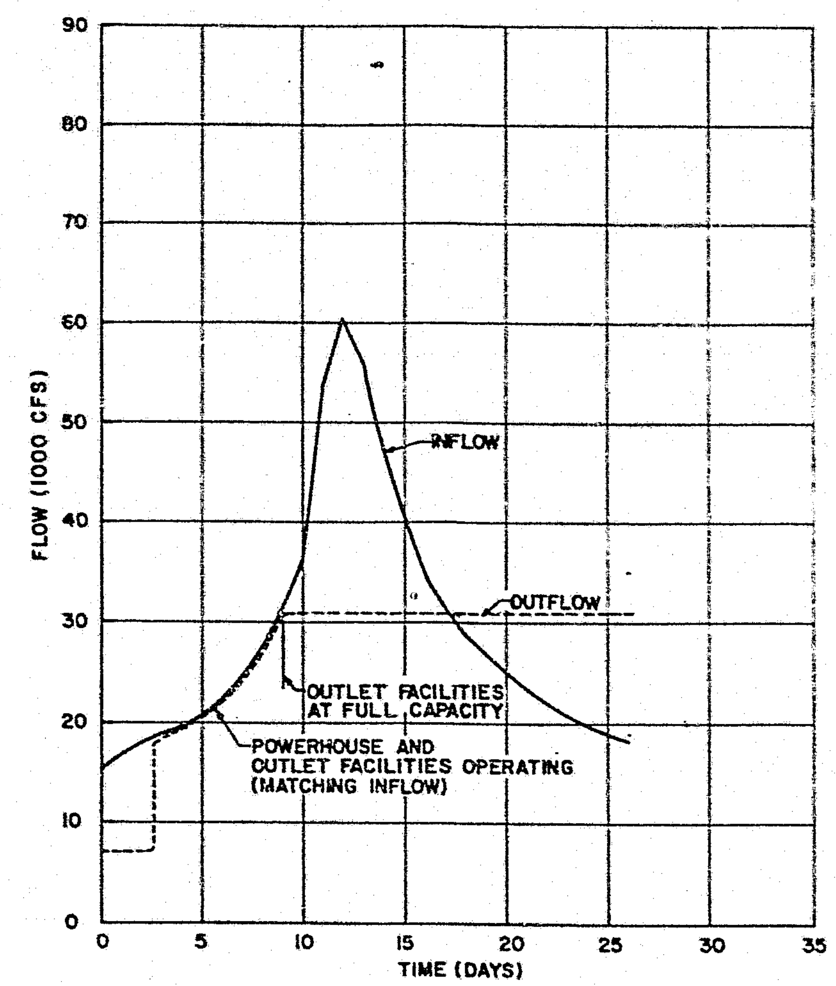
FIGURE B.47

ACRES

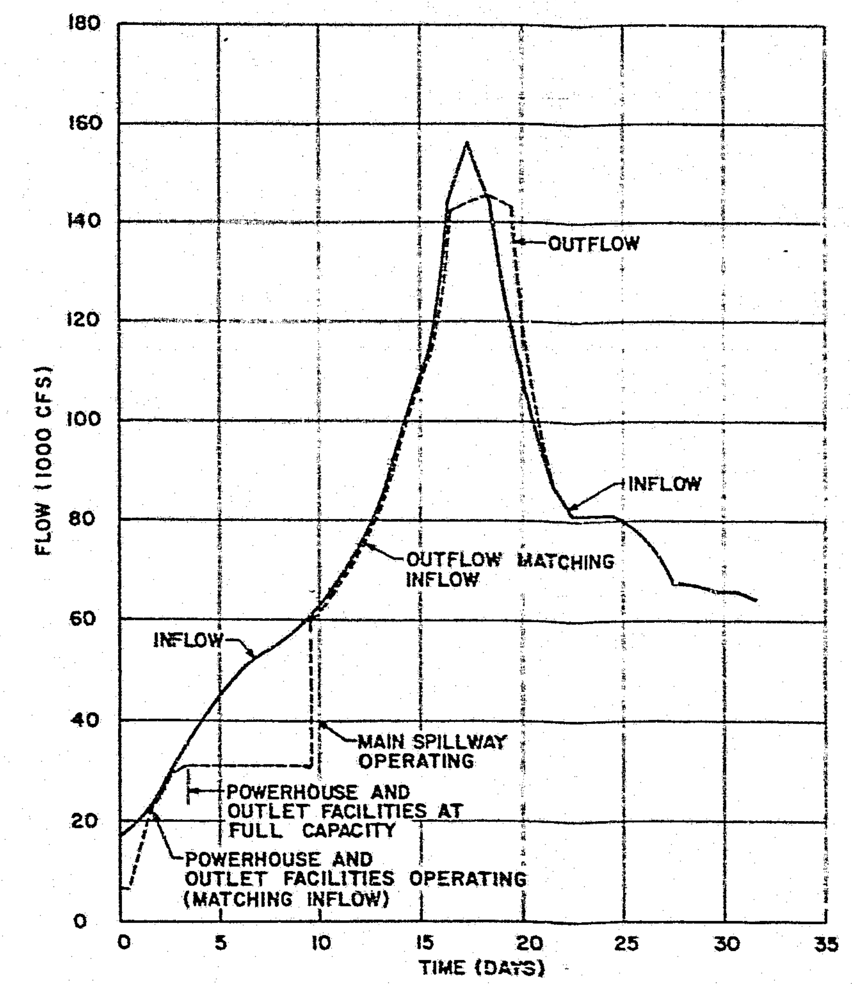


FREQUENCY ANALYSIS OF
AVERAGE ANNUAL ENERGY FOR SUSITNA DEVELOPMENTS

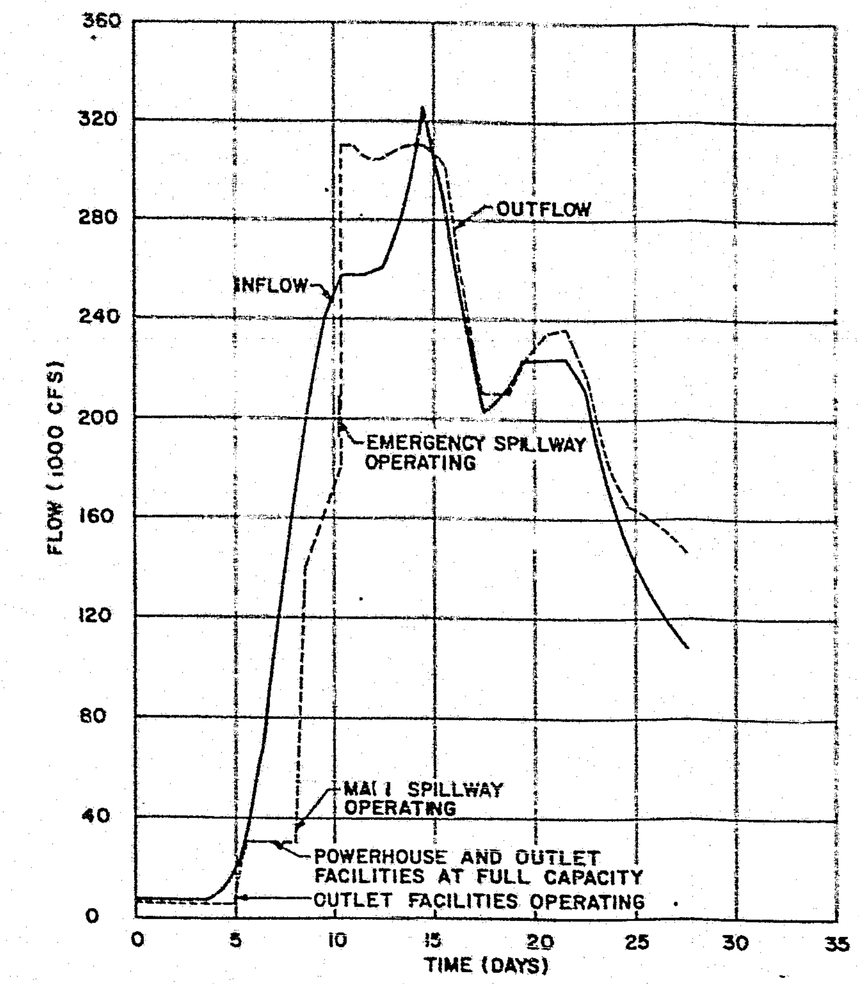
FIGURE B.48



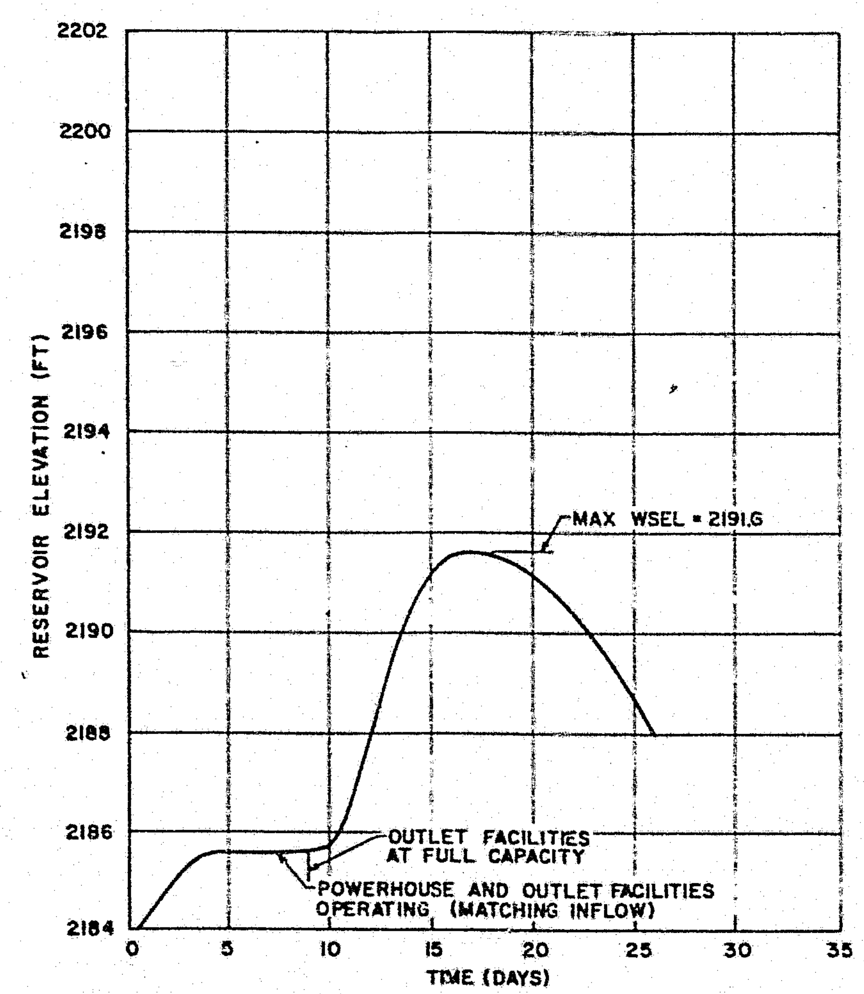
1:50 YEAR FLOOD
(SUMMER)



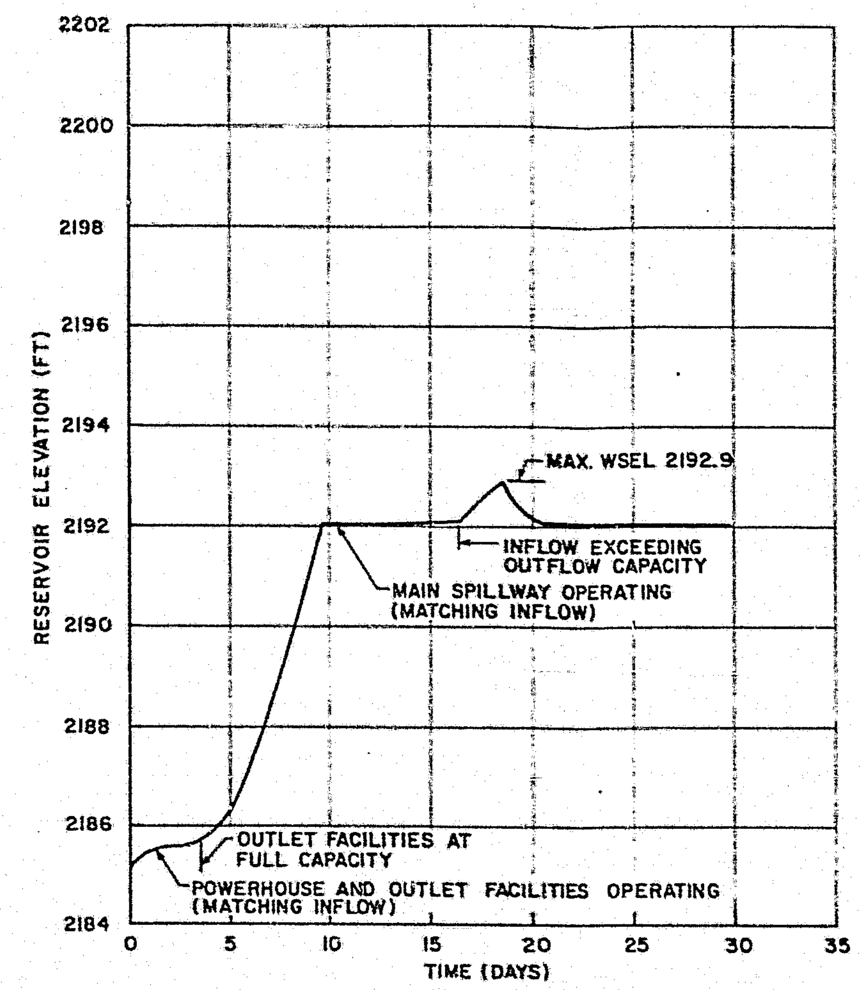
1:10,000 YEAR FLOOD



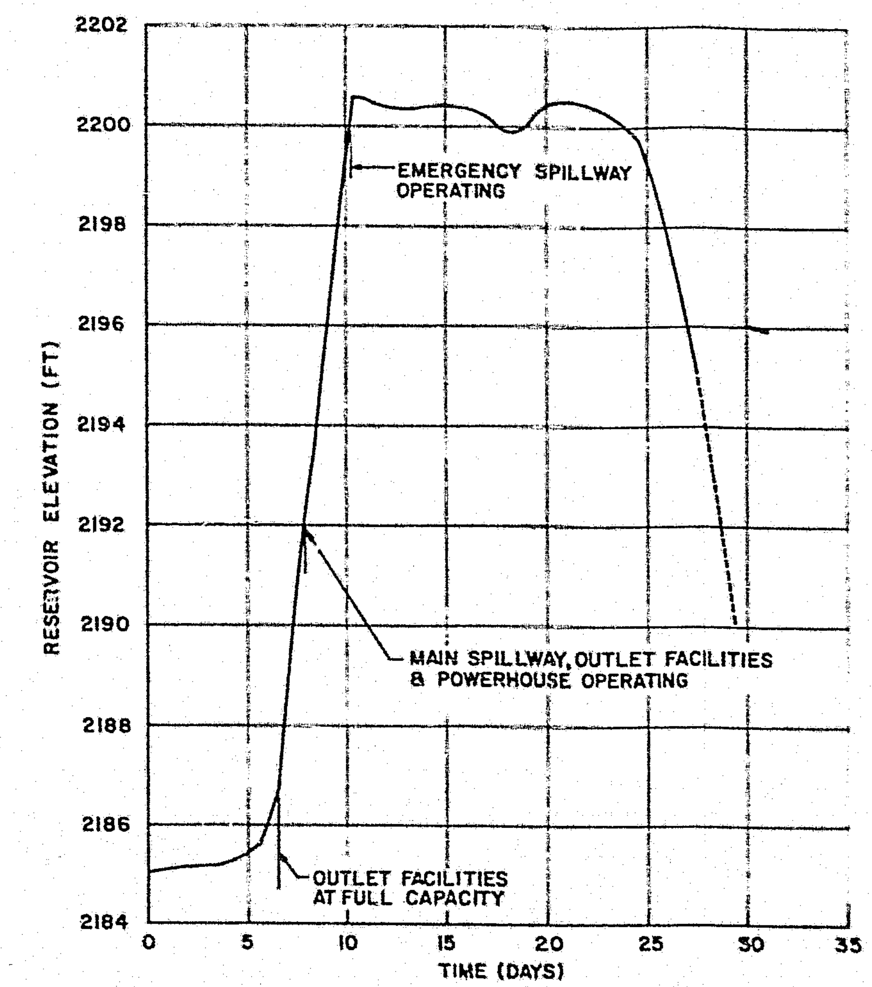
PROBABLE MAXIMUM FLOOD



1:50 YEAR FLOOD
(SUMMER)



1:10,000 YEAR FLOOD



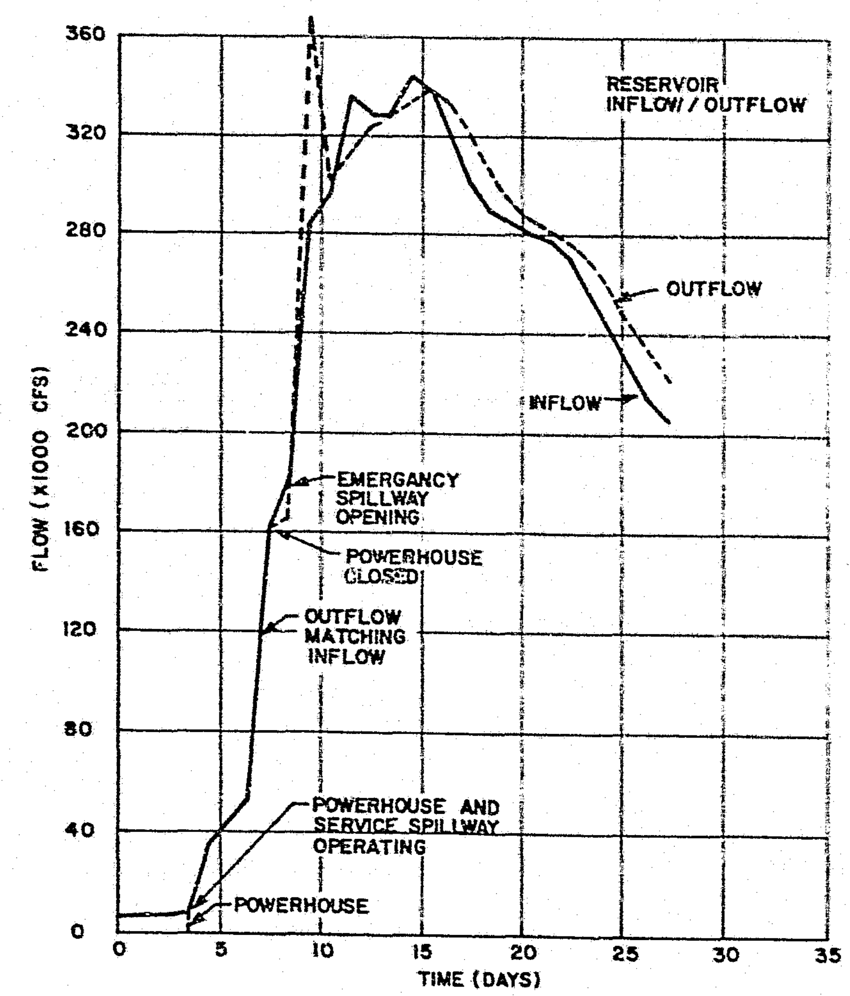
PROBABLE MAXIMUM FLOOD

FIGURE B.49

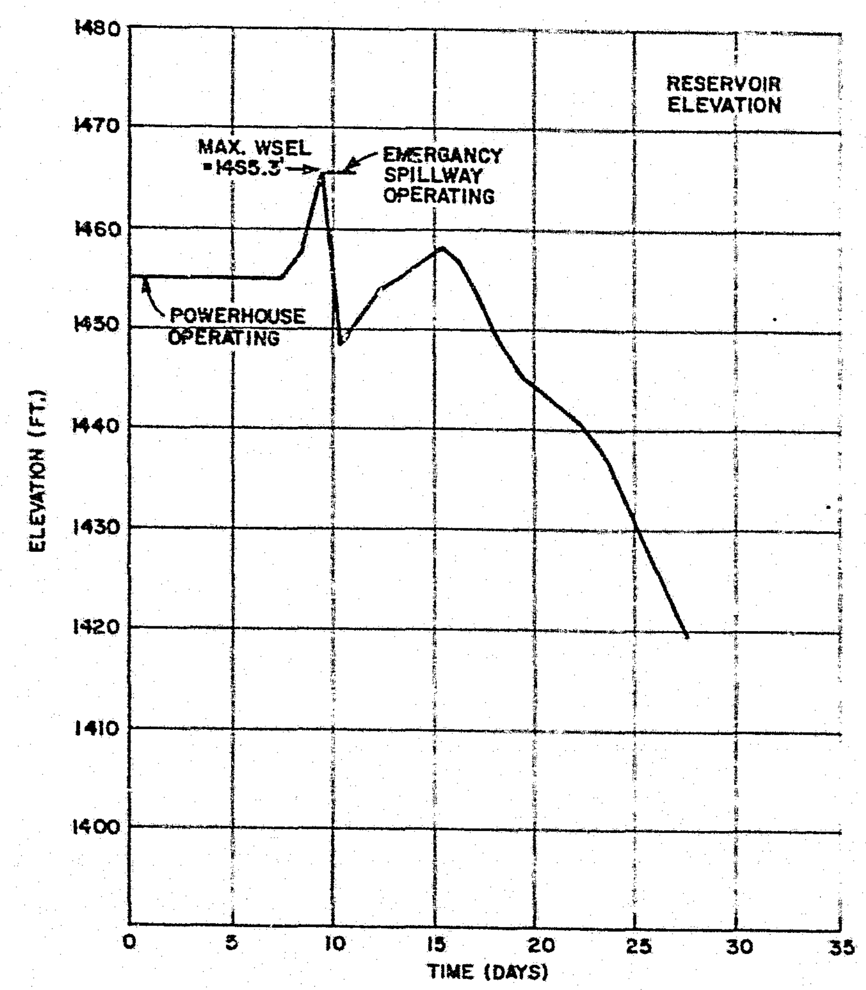
ACRES | ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

WATANA
HYDROLOGICAL DATA
SHEET 2

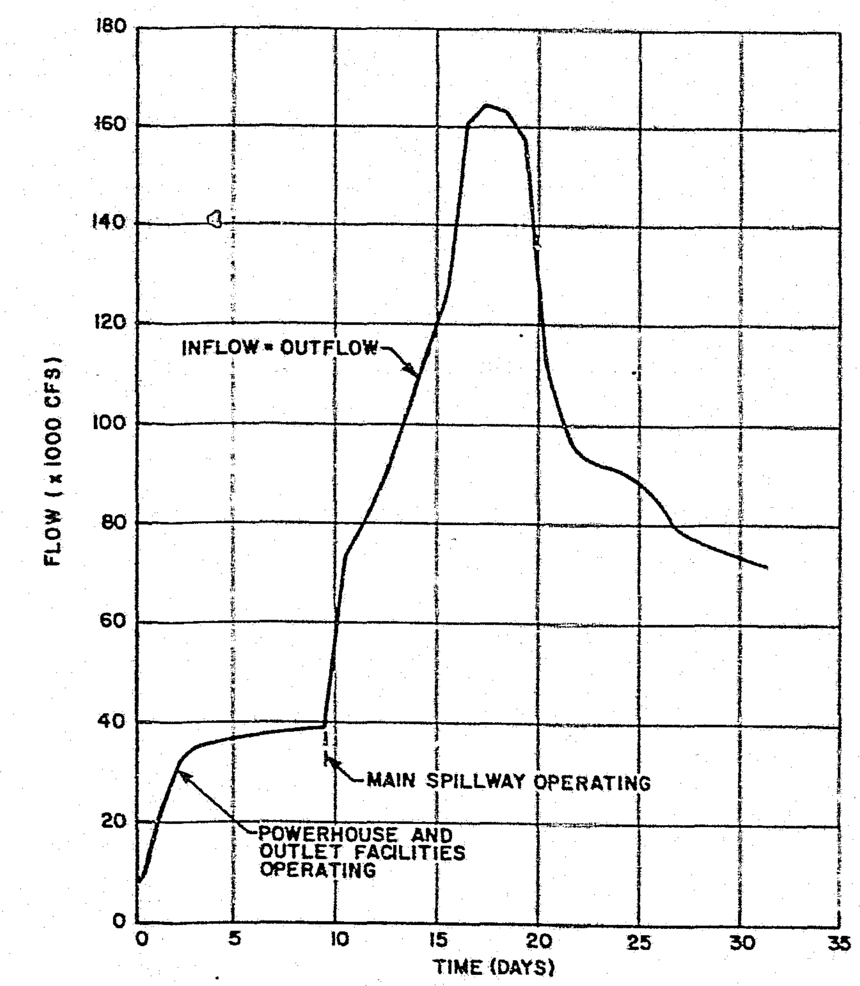
ACRES AMERICAN INCORPORATED | MARCH 1982



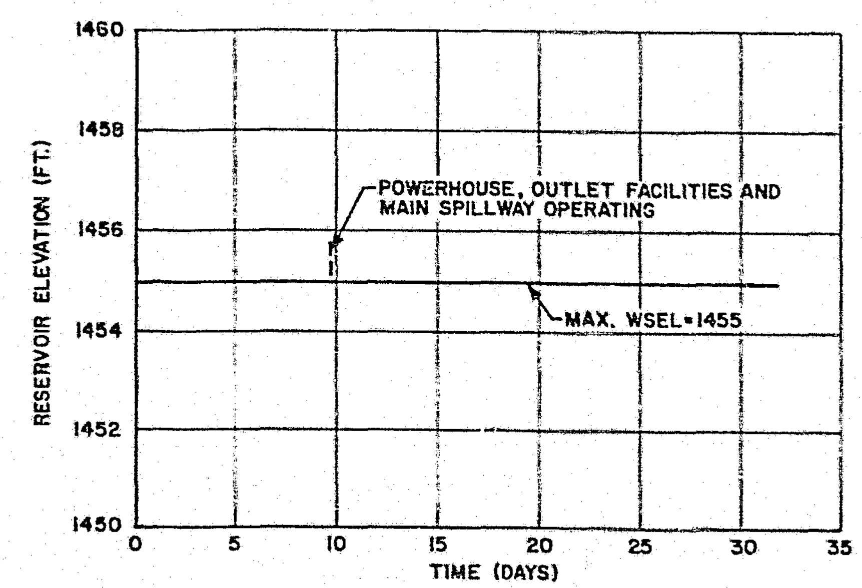
PROBABLE MAXIMUM FLOOD



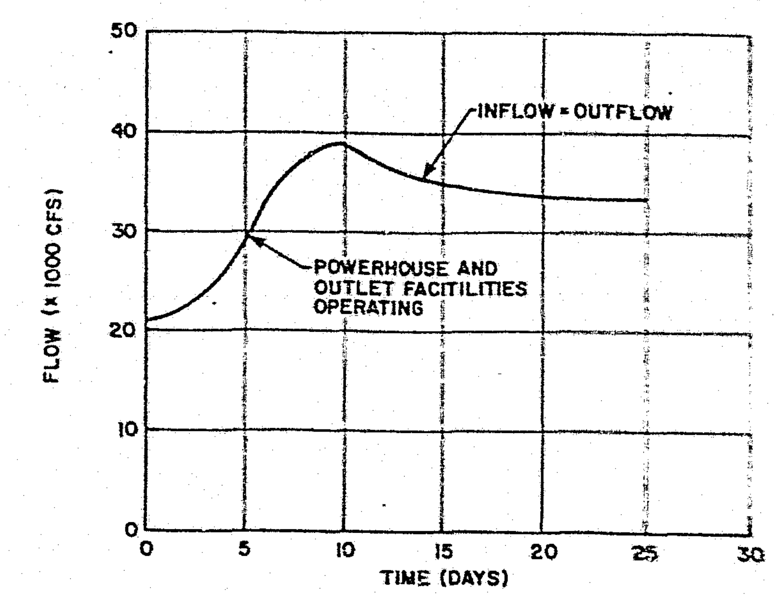
PROBABLE MAXIMUM FLOOD



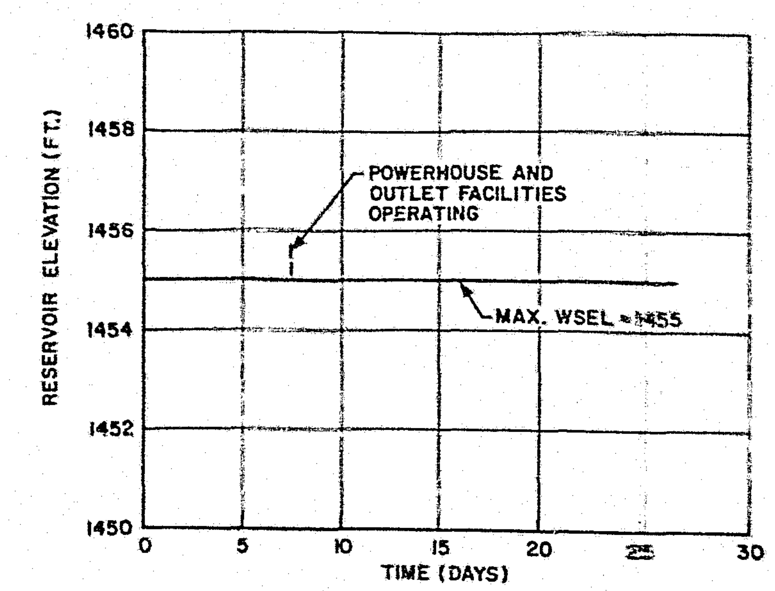
RESERVOIR ROUTING
1:10,000 YR. FLOOD



RESERVOIR ROUTING
1:10,000 YR. FLOOD



RESERVOIR ROUTING
1:50 YR. SUMMER FLOOD



RESERVOIR ROUTING
1:50 YR. SUMMER FLOOD

FIGURE B.50

ACRES

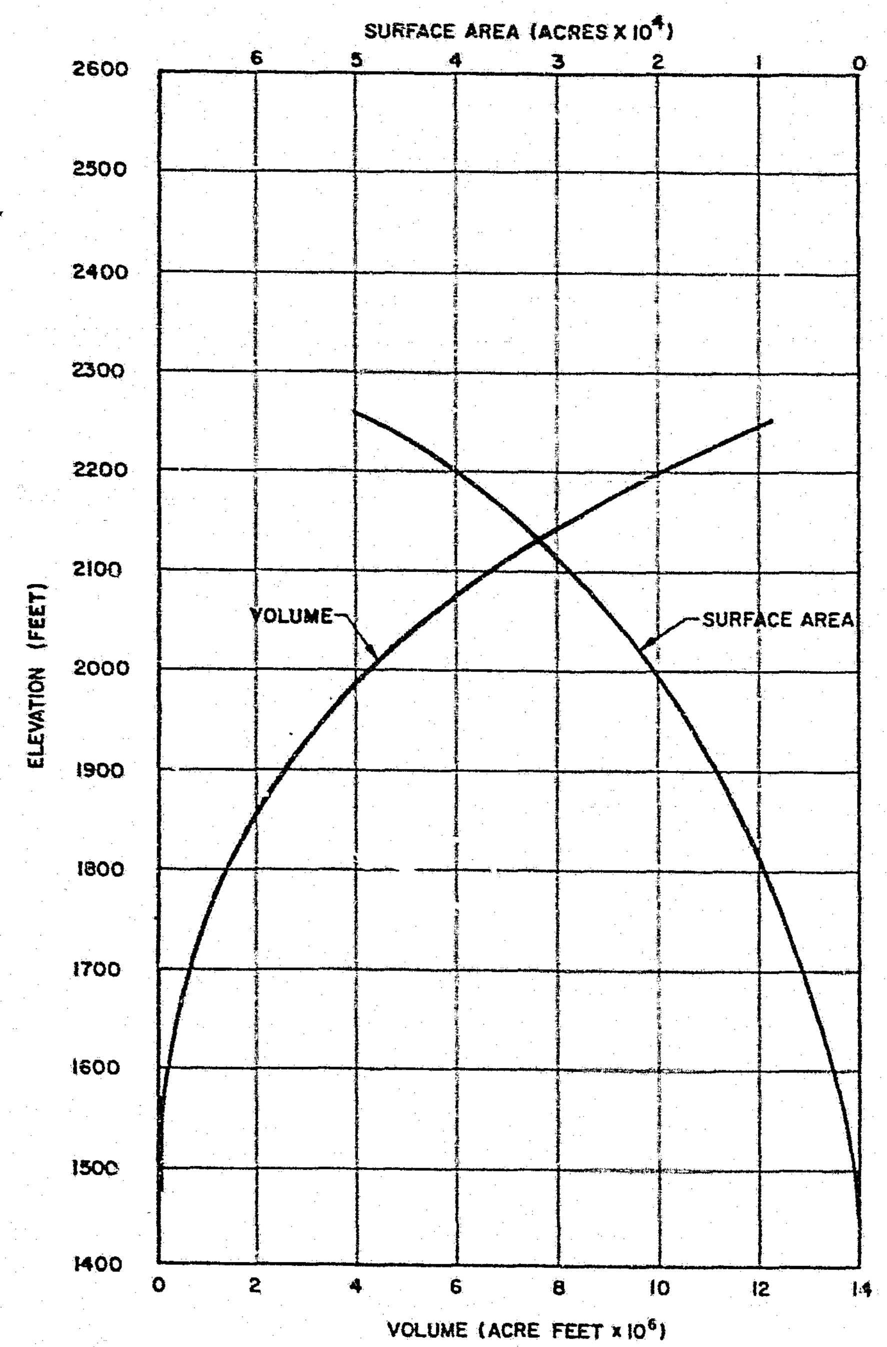
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

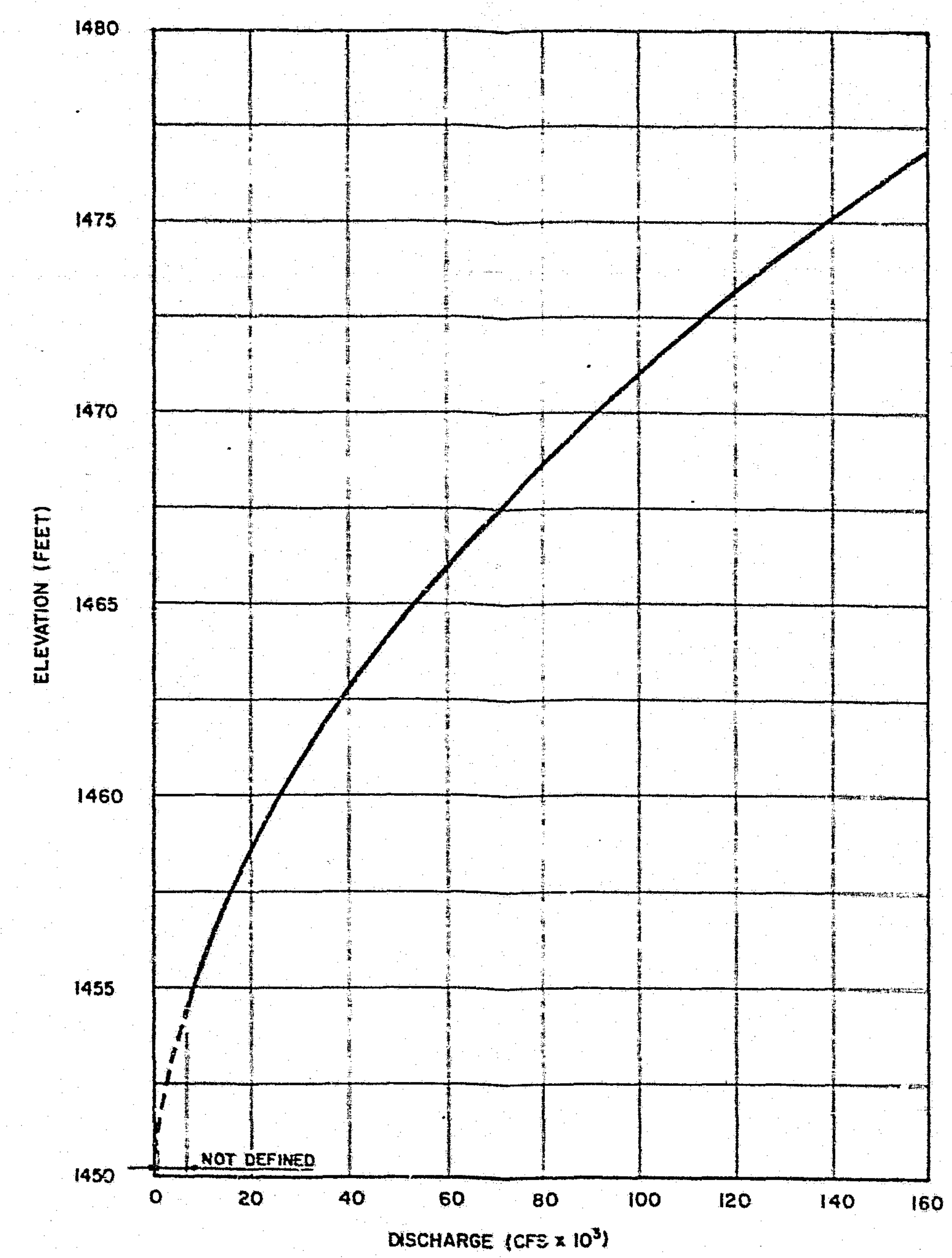
DEVIL CANYON
HYDROLOGICAL DATA
SHEET 2

ACRES AMERICAN INCORPORATED

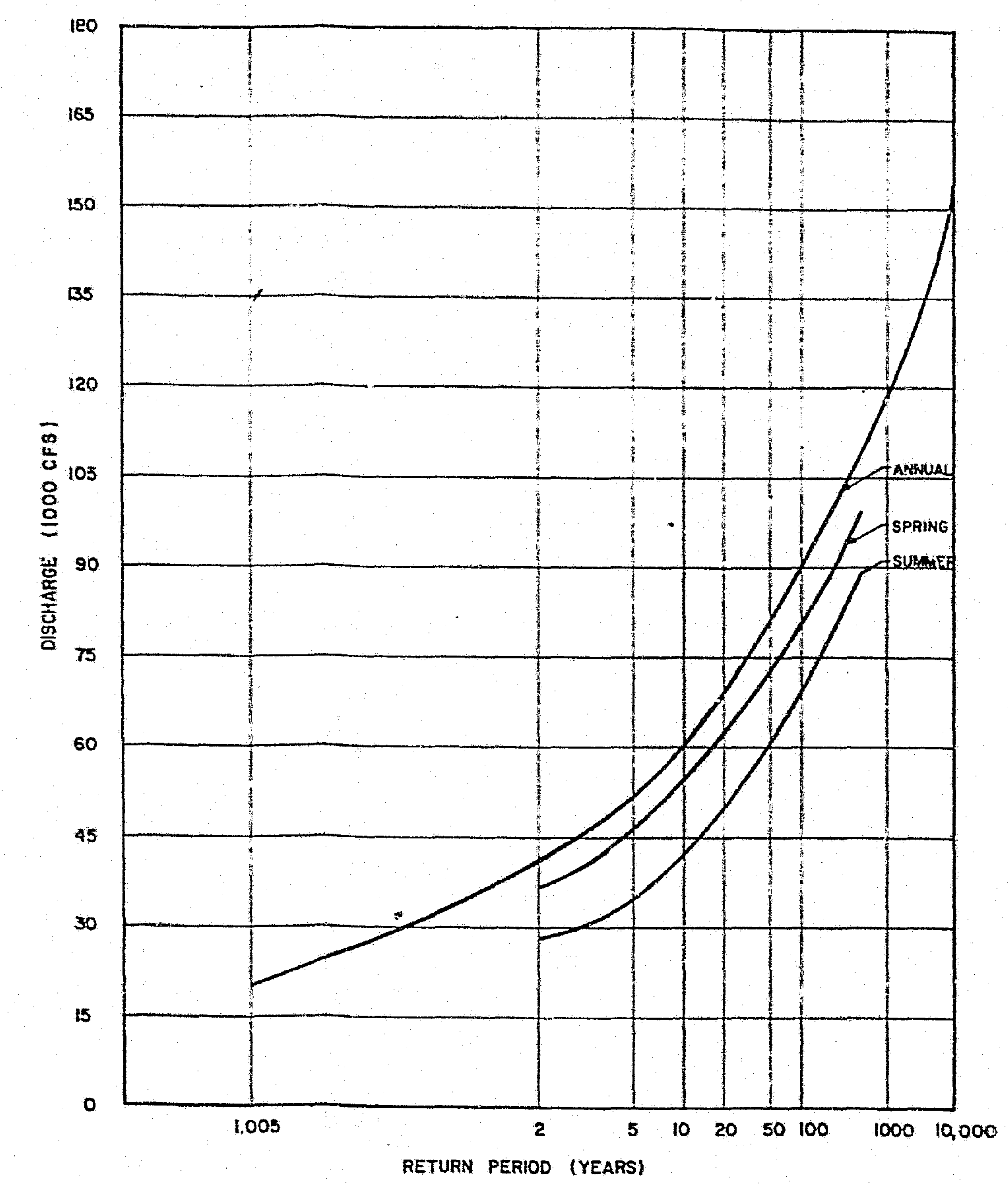
MARCH 1982



RESERVOIR VOLUME AND SURFACE AREA



TAILWATER RATING



INFLOW FLOOD FREQUENCY

FIGURE B.51

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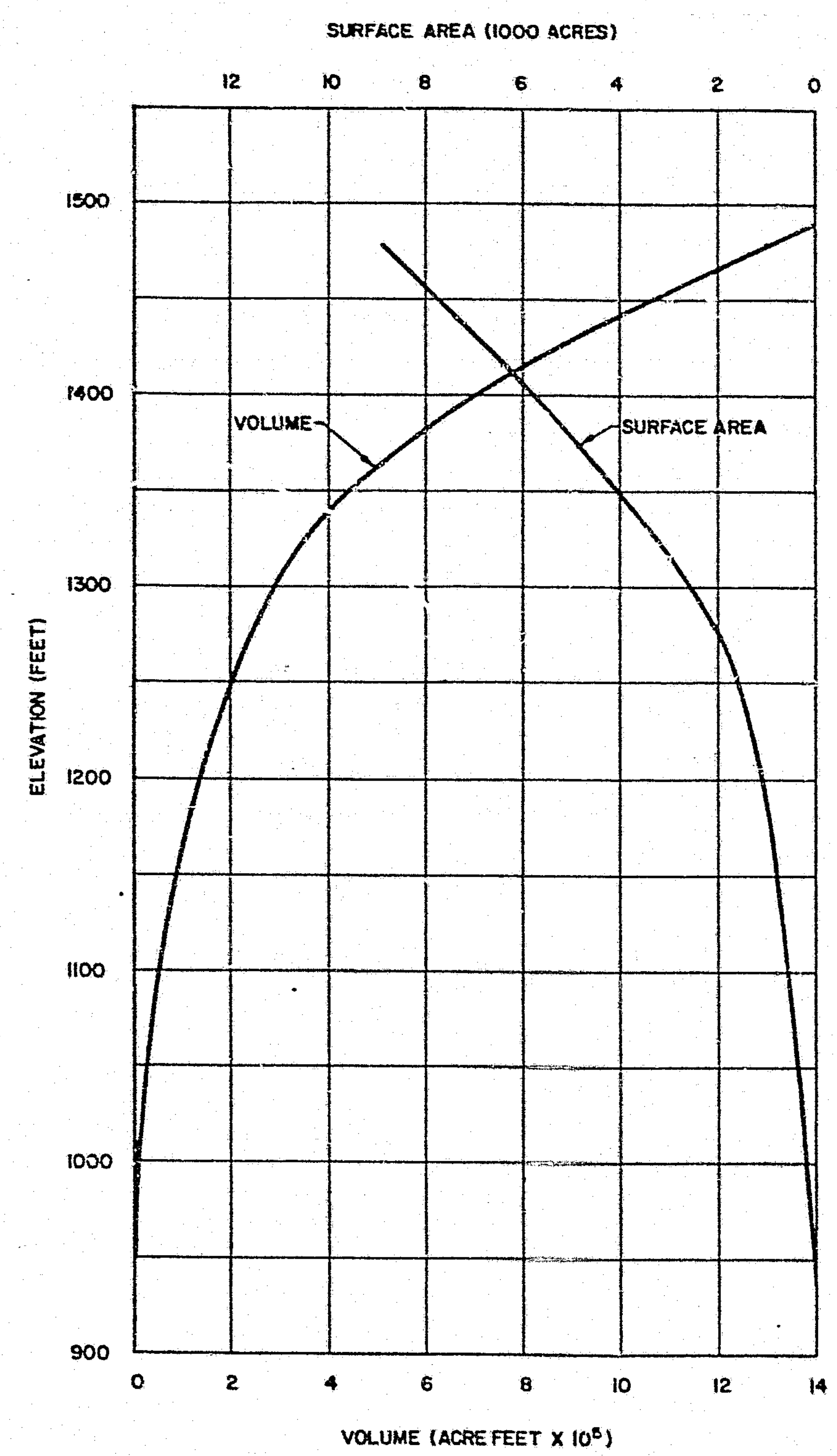
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

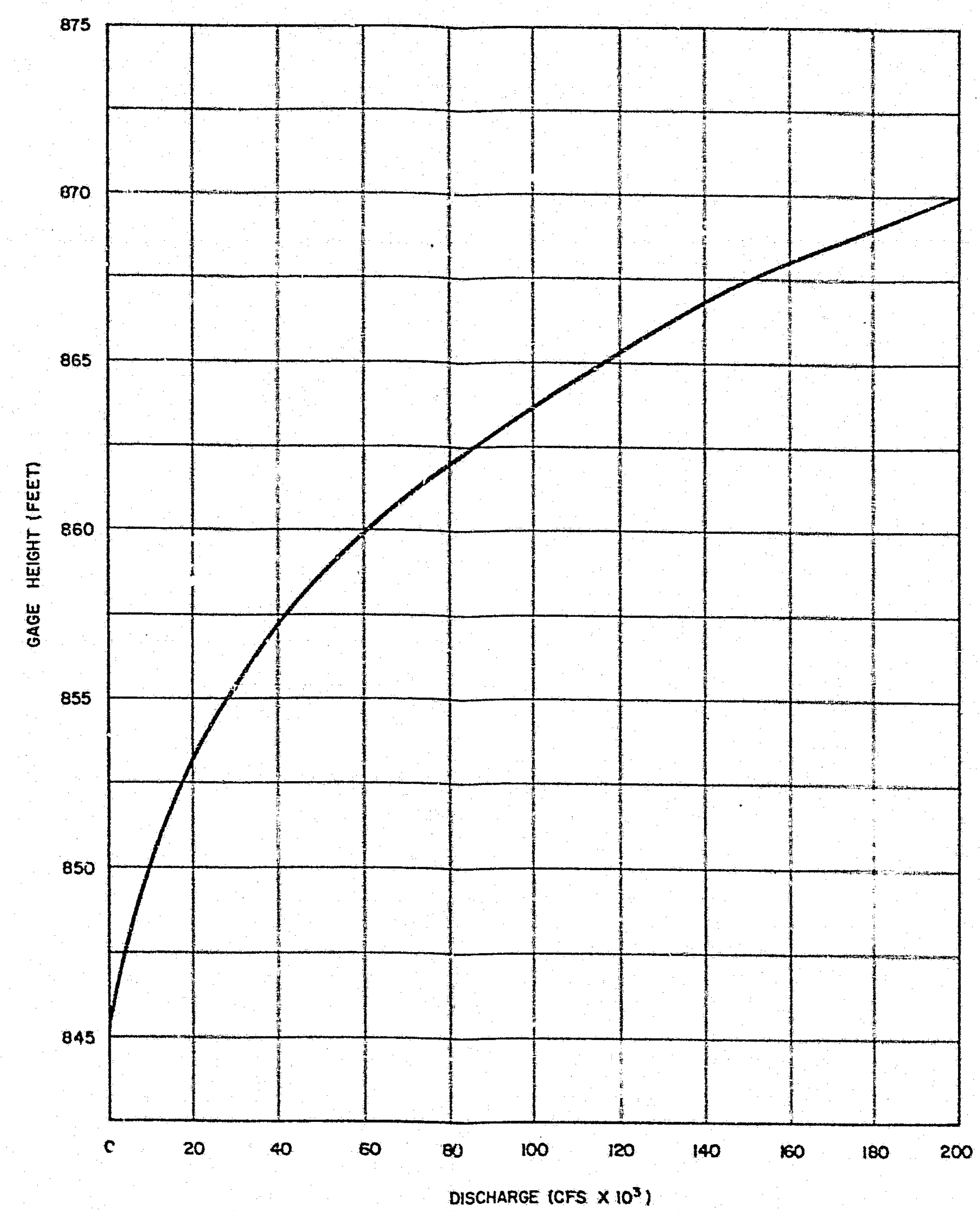
WATANA
HYDRLOGICAL DATA
SHEET I

ACRES AMERICAN INCORPORATED

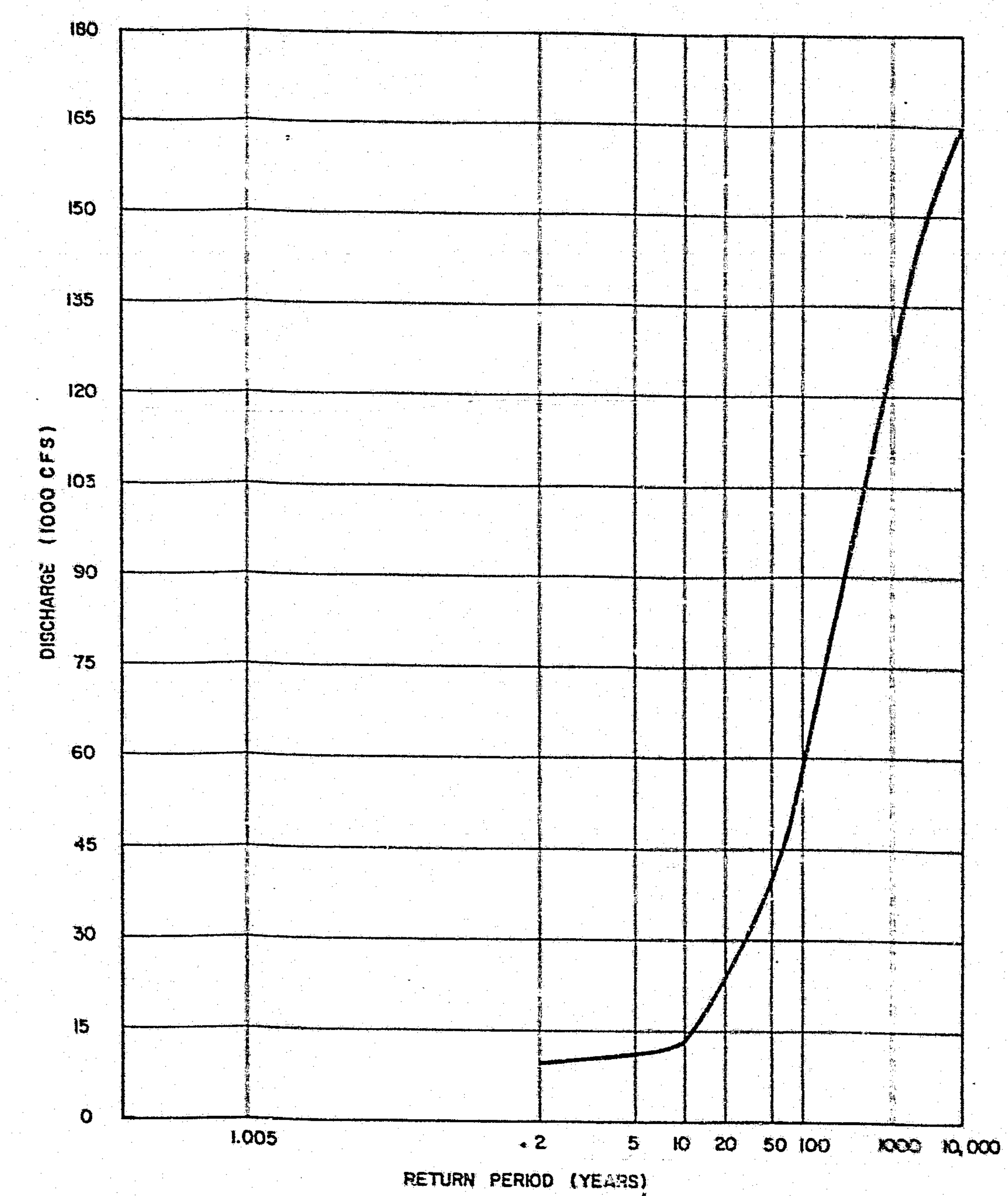
MARCH 1982



RESERVOIR VOLUME AND SURFACE AREA



TAILWATER RATING CURVE



FLOOD FREQUENCY CURVE
(INFLOW AFTER ROUTING THROUGH WATANA)

FIGURE B.52

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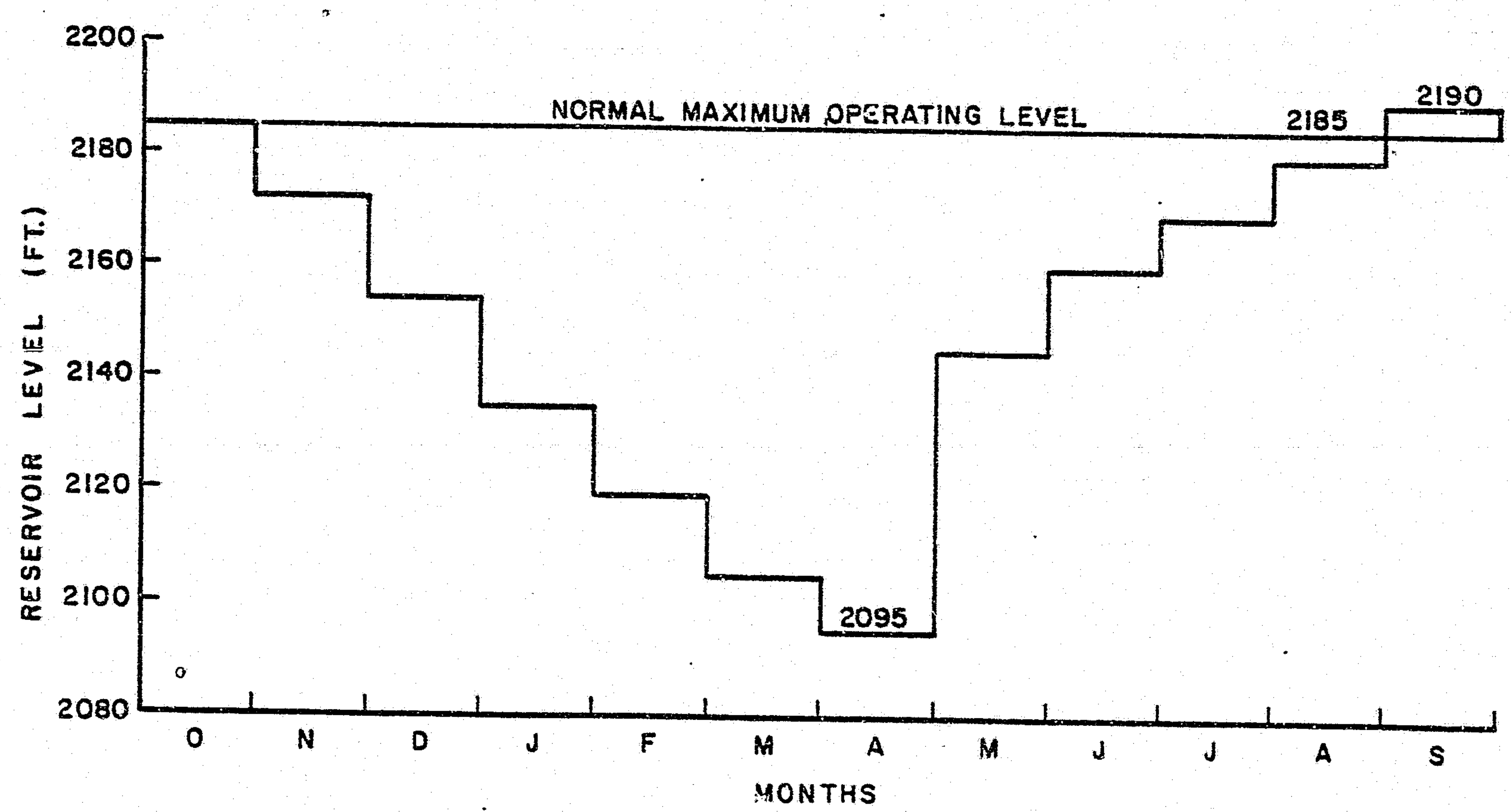
ALASKA POWER AUTHORITY

SUSITNA HYDROELECTRIC PROJECT

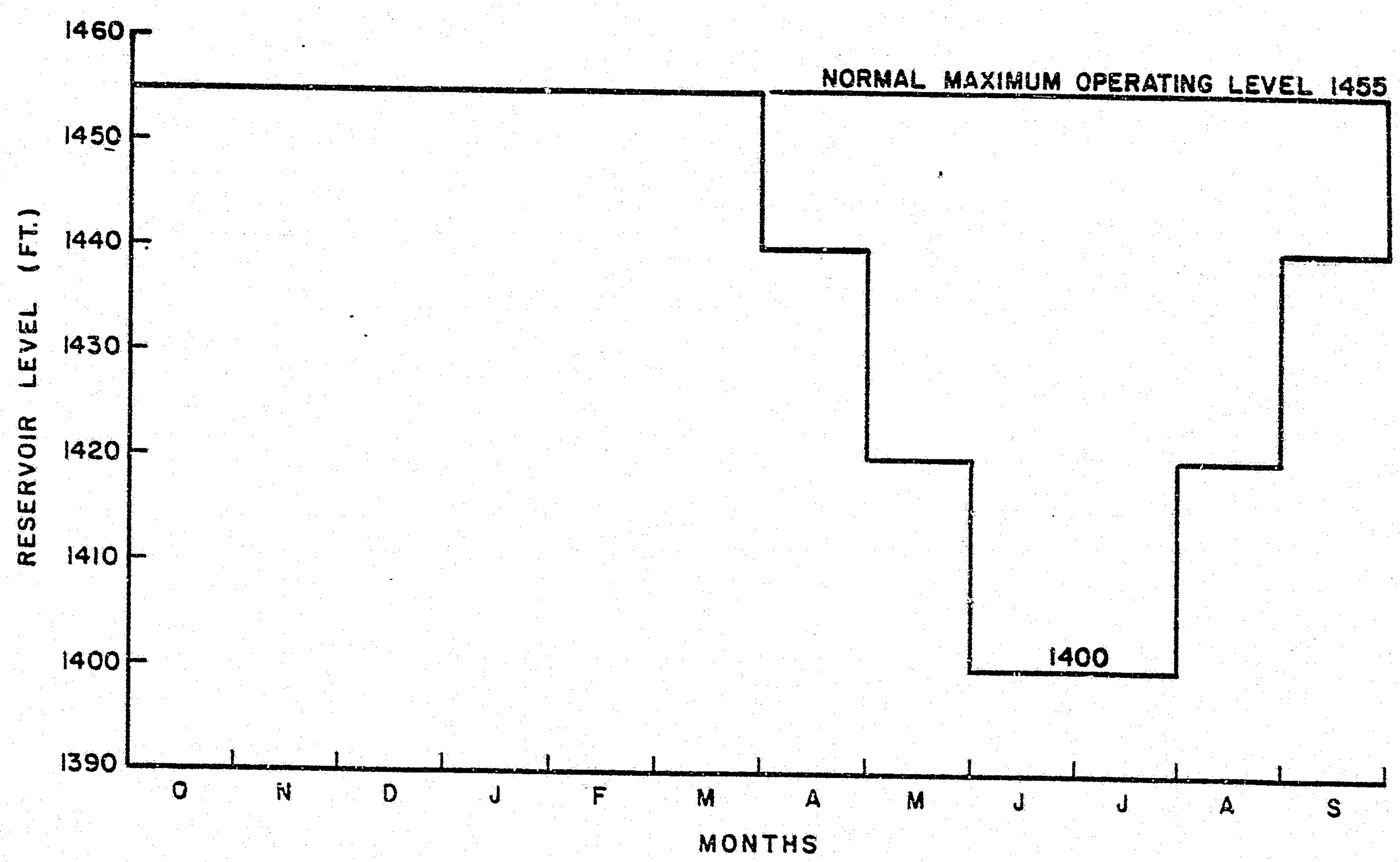
DEVIL CANYON
HYDROLOGICAL DATA
SHEET I

ACRES AMERICAN INCORPORATED

MARCH 1982



WATANA RESERVOIR

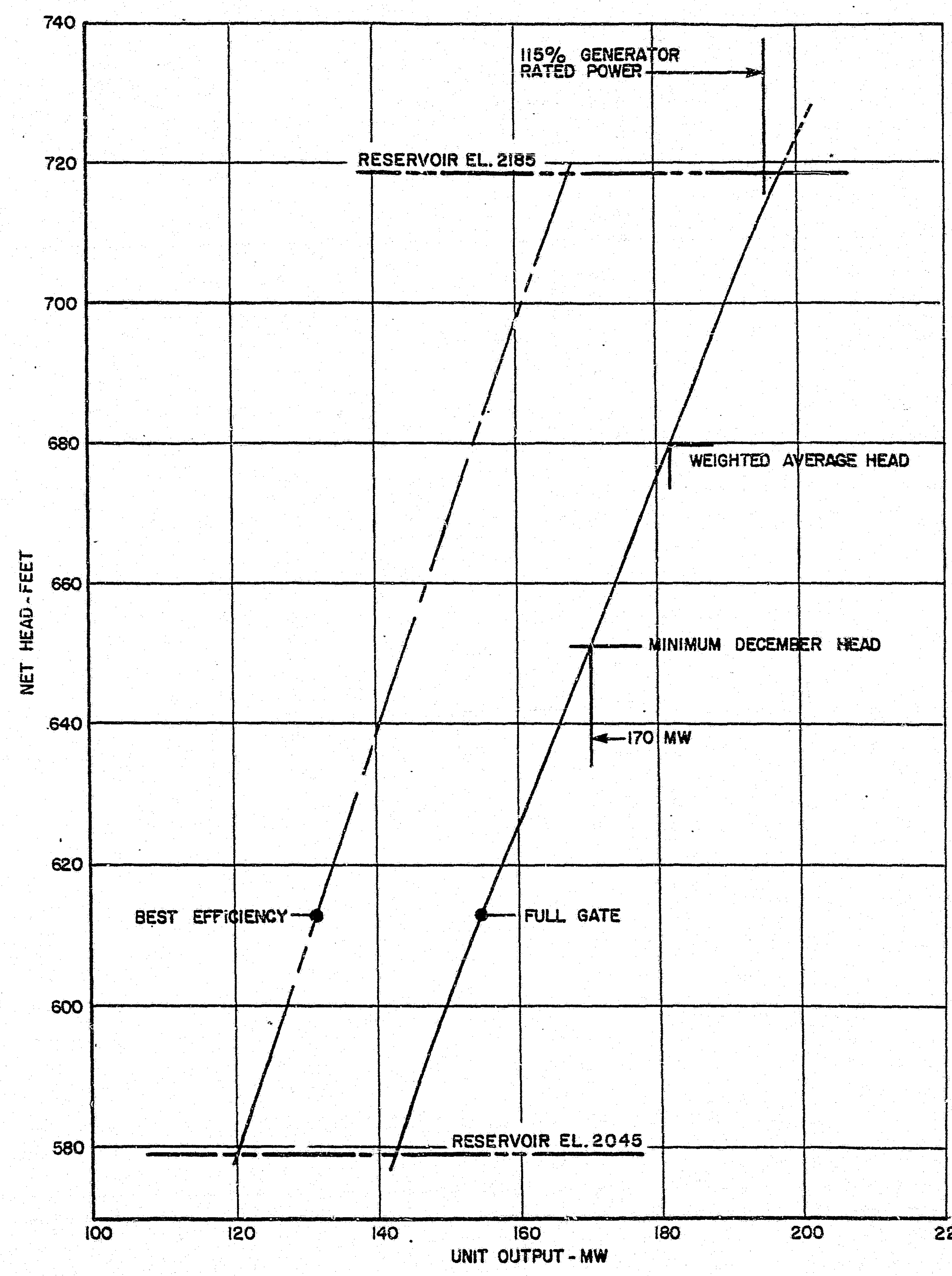


DEVIL CANYON RESERVOIR

MONTHLY TARGET MINIMUM RESERVOIR LEVELS

ACRES

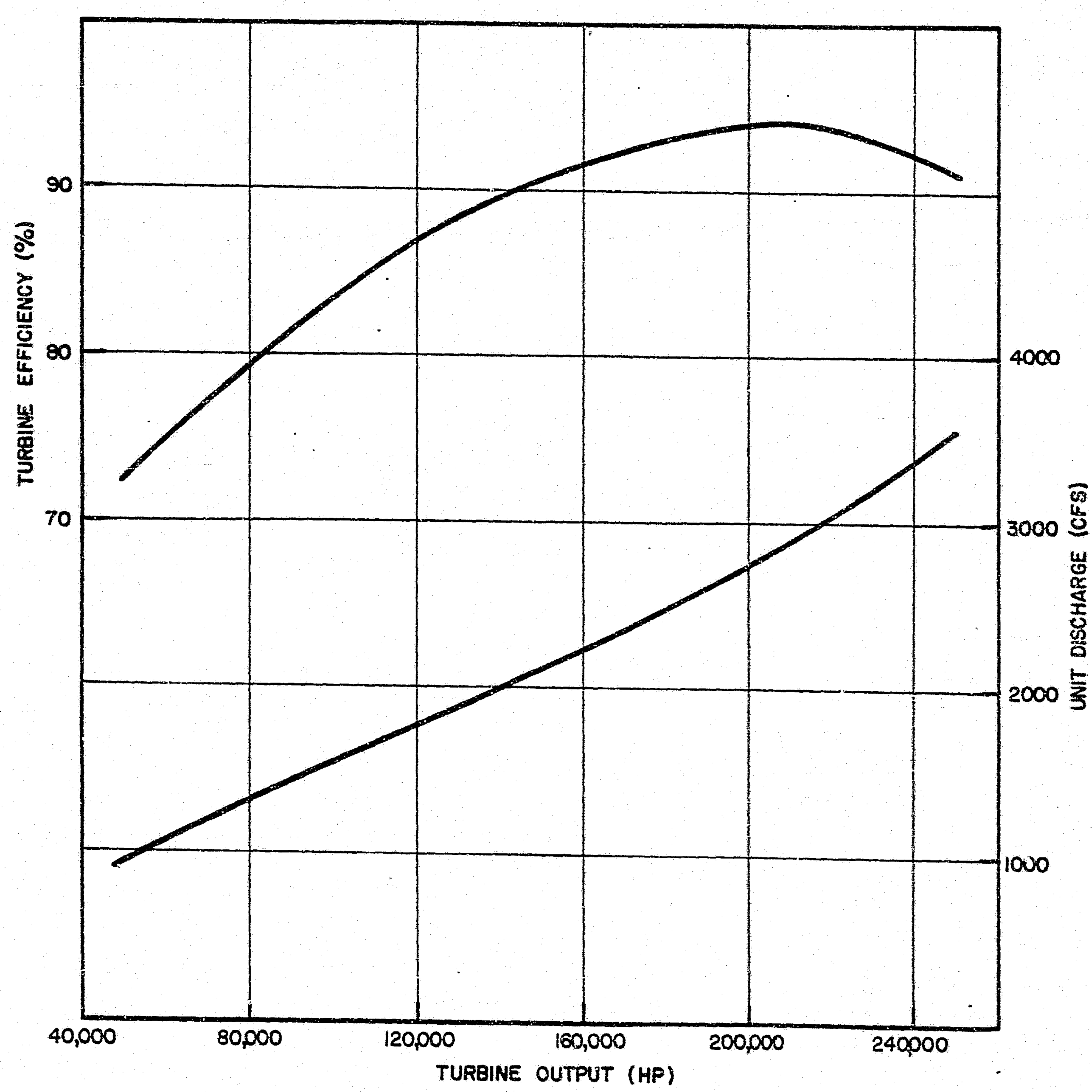
FIGURE B.53



WATANA - UNIT OUTPUT

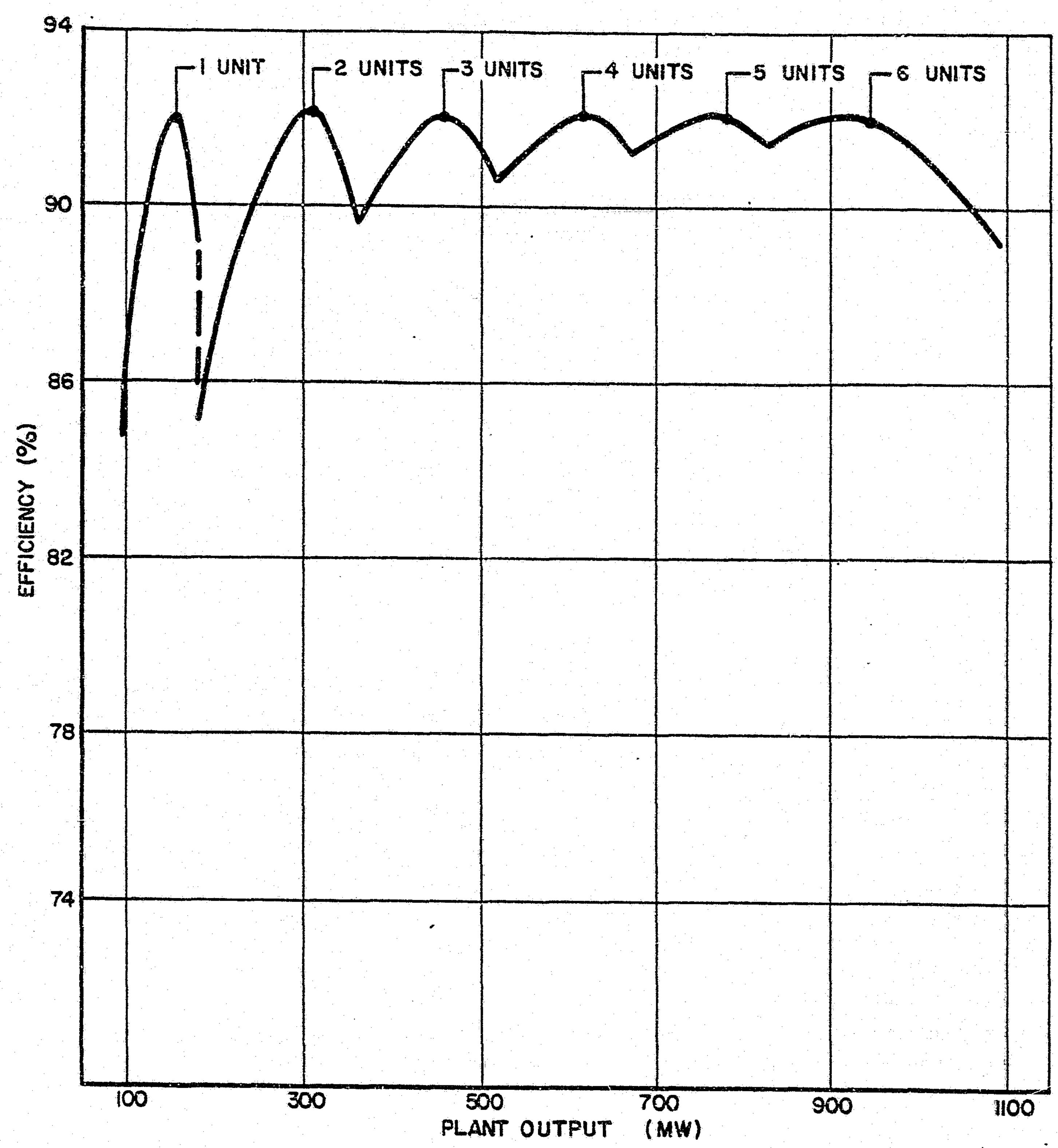
ACRES

FIGURE B.54



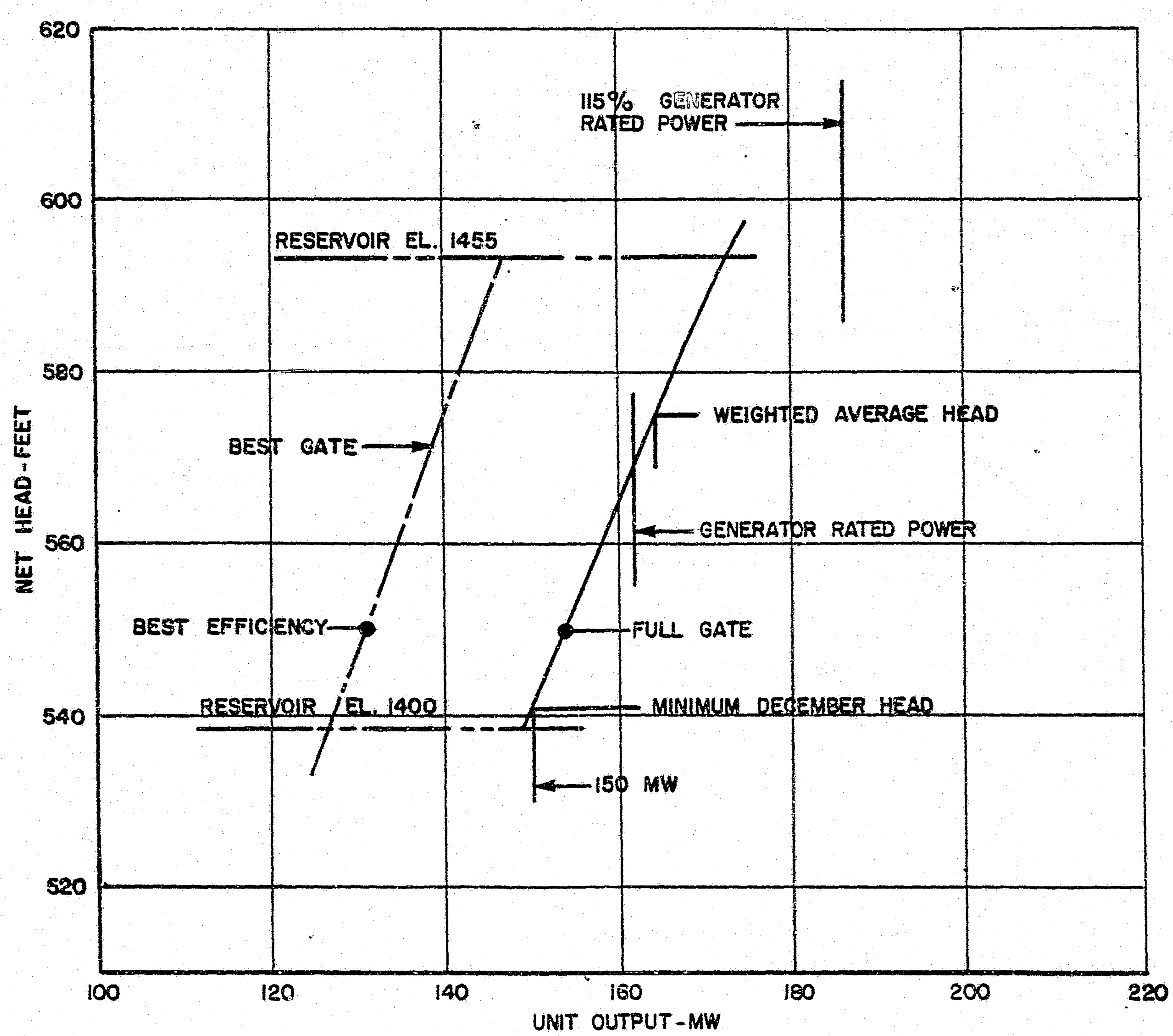
WATANA - TURBINE PERFORMANCE
(AT RATED HEAD)

FIGURE B.55



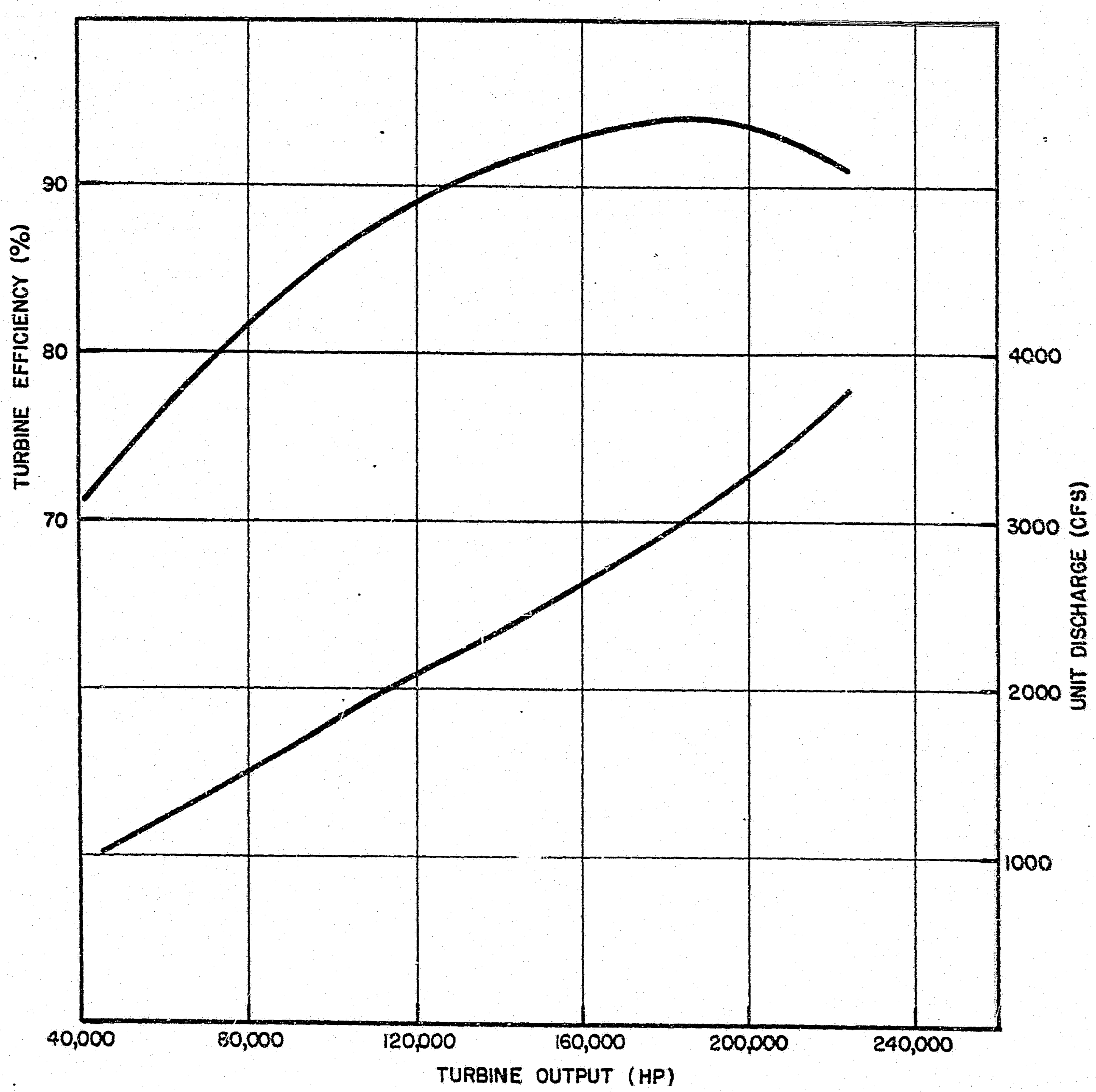
WATANA-UNIT EFFICIENCY
(AT RATED HEAD)

FIGURE B.53



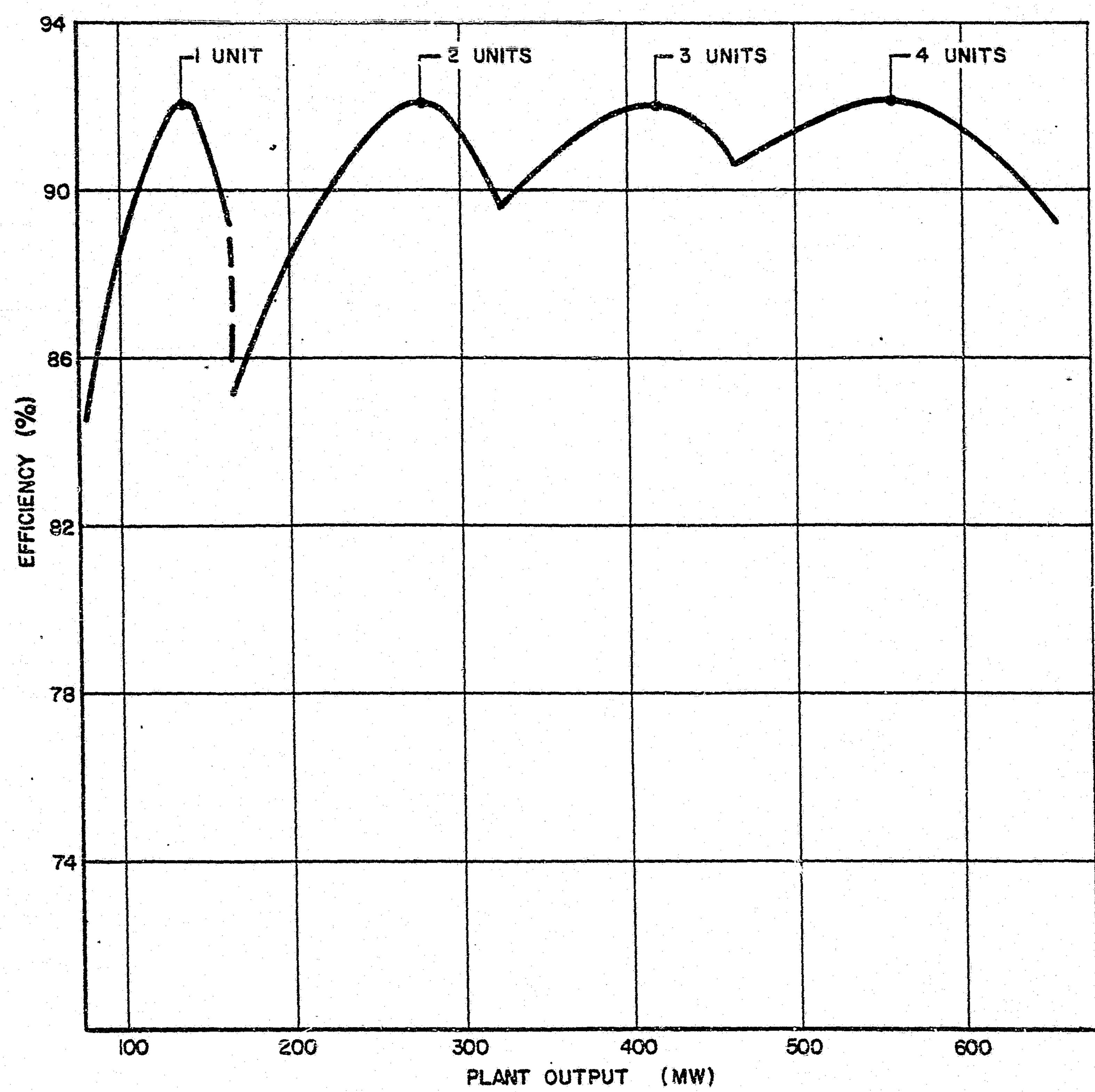
DEVIL CANYON-UNIT OUTPUT

FIGURE B.57



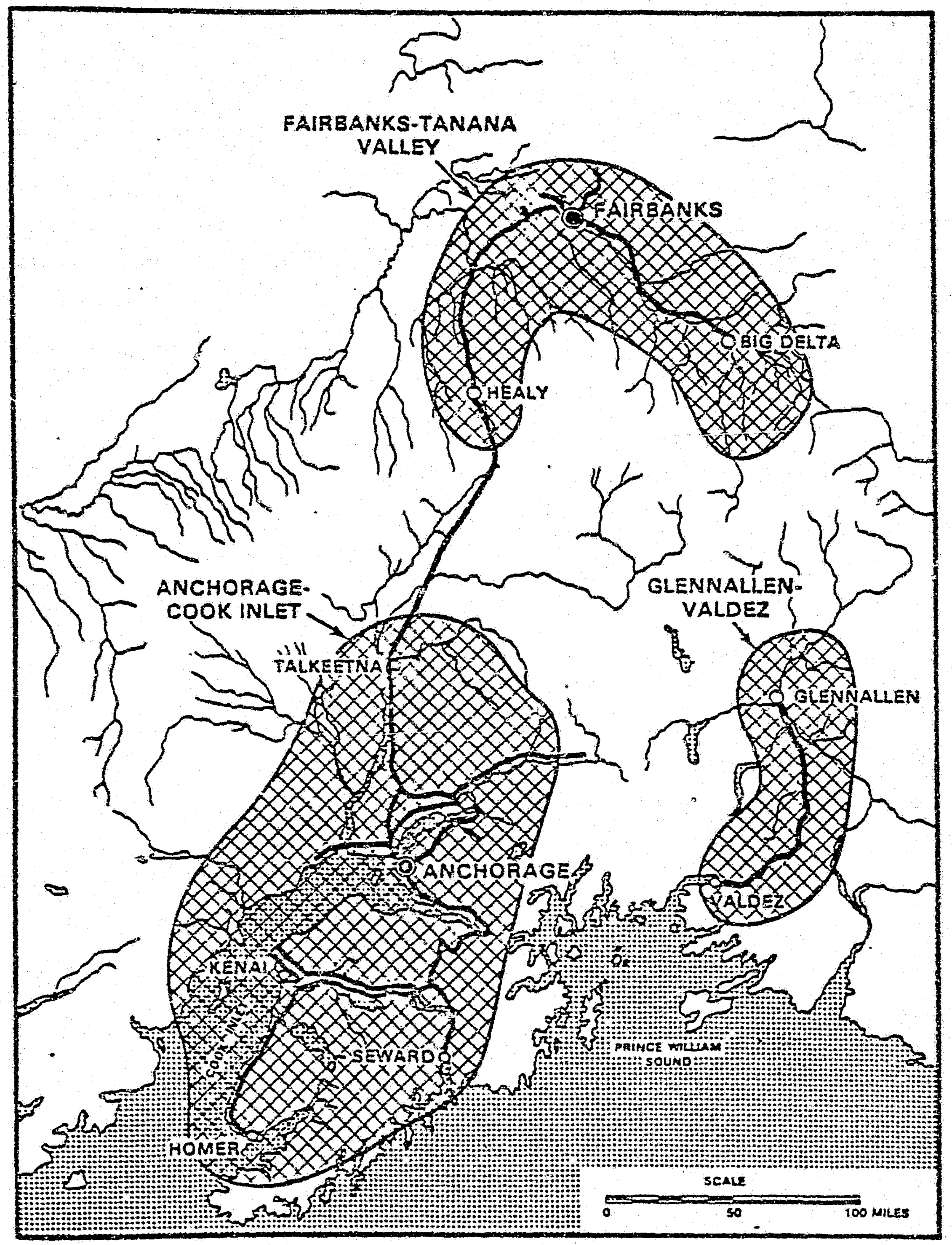
DEVIL CANYON - TURBINE PERFORMANCE
(AT RATED HEAD)

FIGURE B.58



DEVIL CANYON - UNIT EFFICIENCY
(AT RATED HEAD)

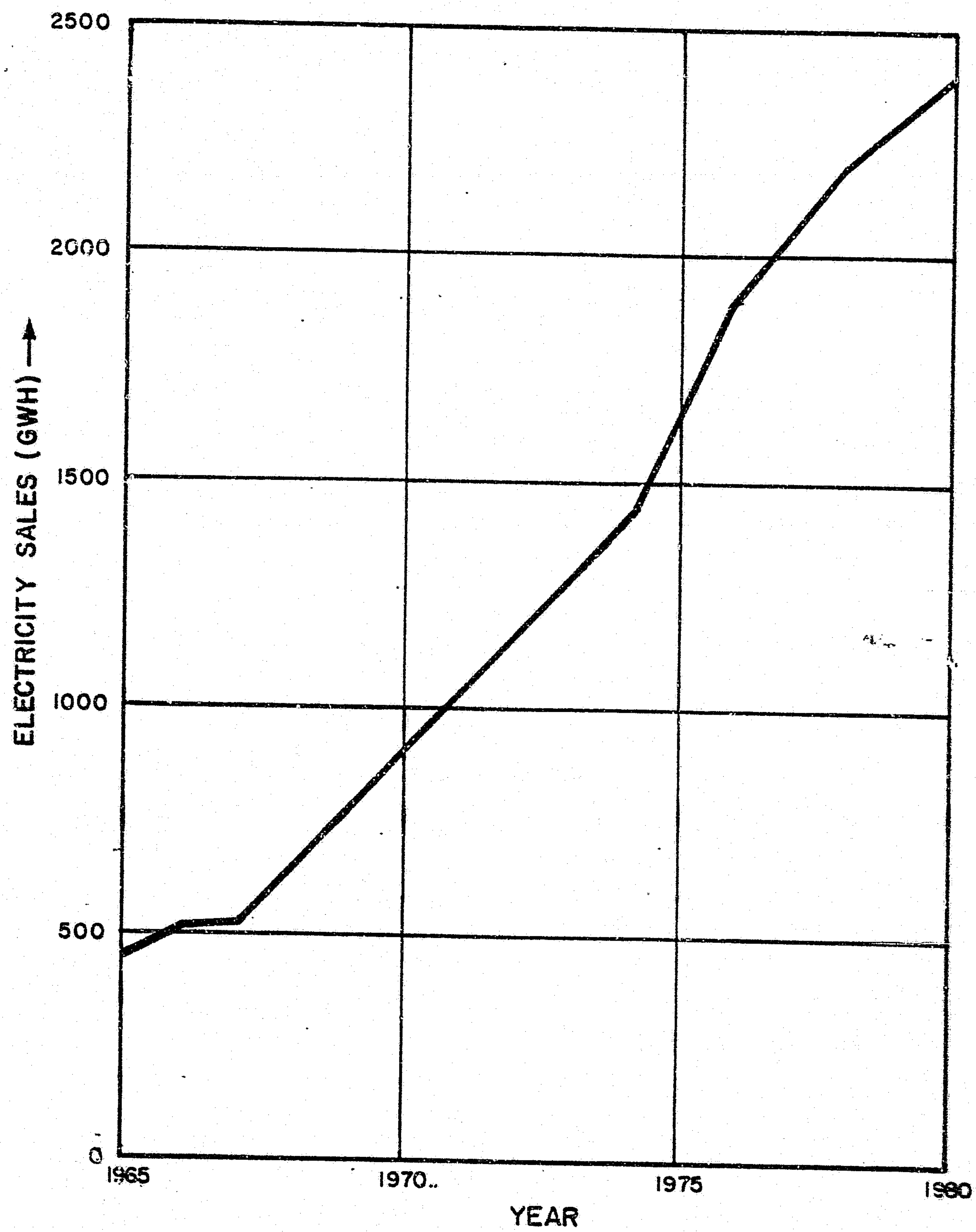
FIGURE B.59



RAILBELT AREA OF ALASKA
SHOWING ELECTRICAL LOAD CENTERS

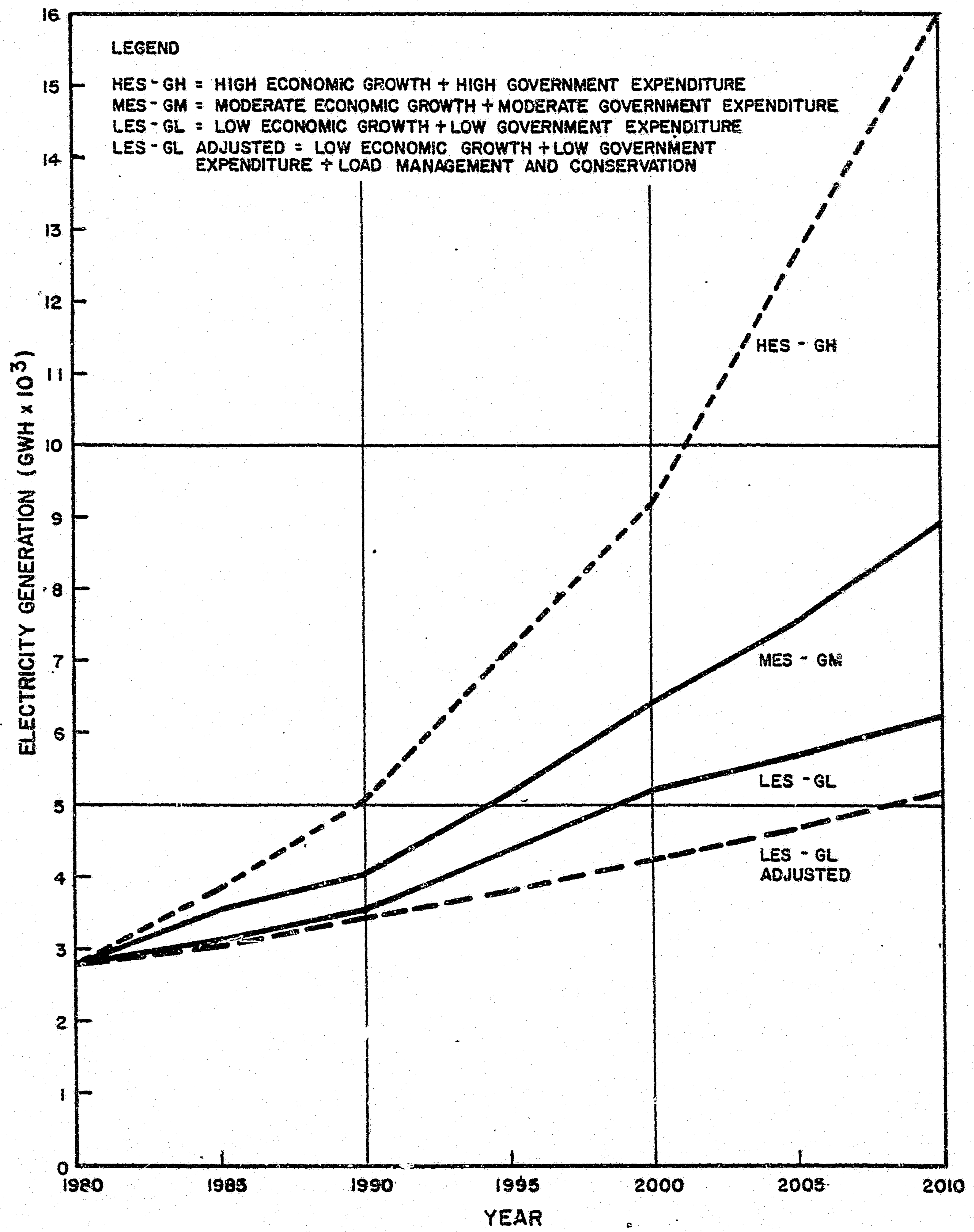
FIGURE B.60

ACRES



HISTORICAL TOTAL RAILBELT UTILITY SALES
TO FINAL CUSTOMERS

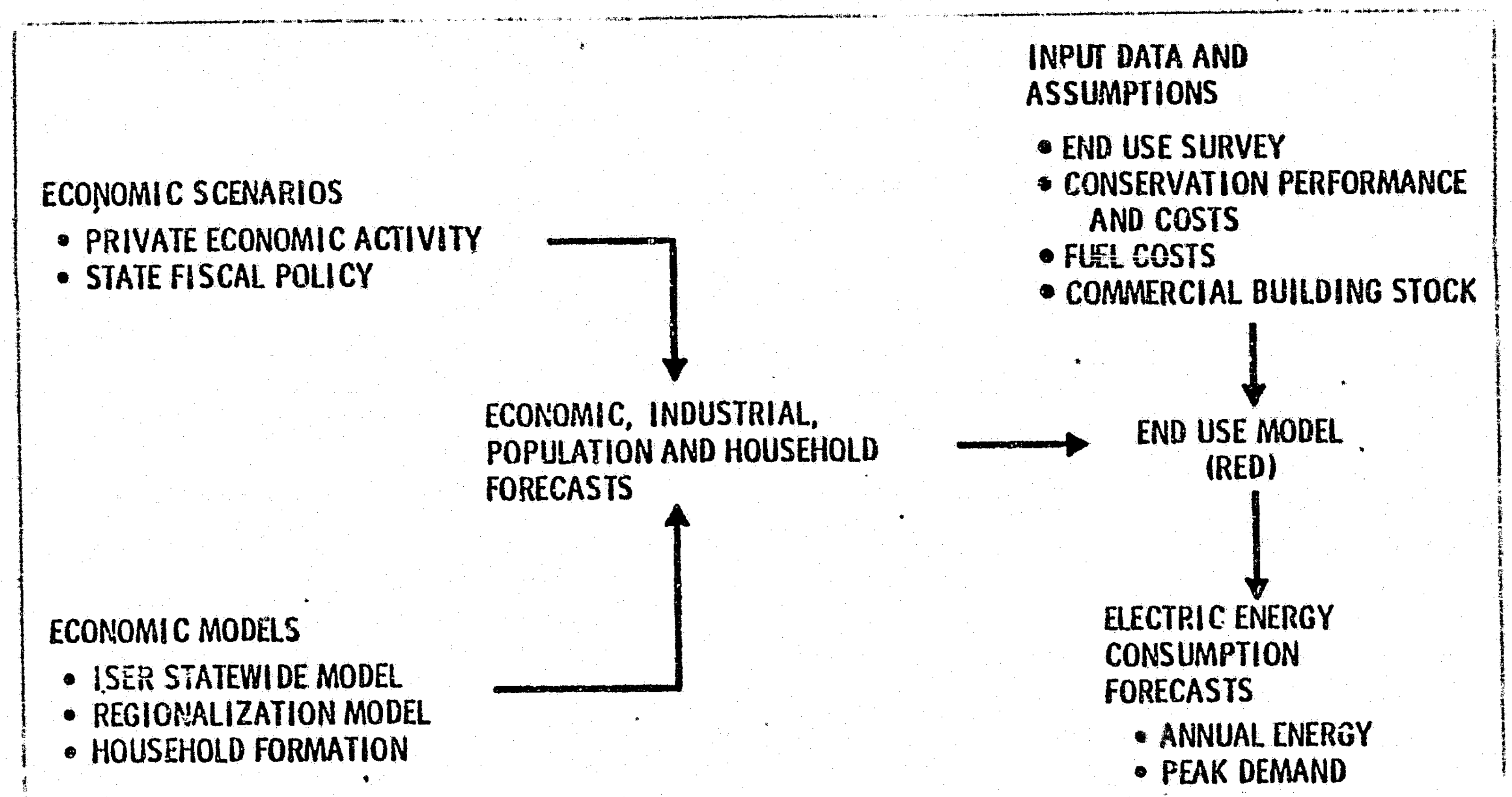
FIGURE B.61



ISER 1980 ENERGY FORECASTS USED
FOR DEVELOPMENT SELECTION STUDIES

ACRES

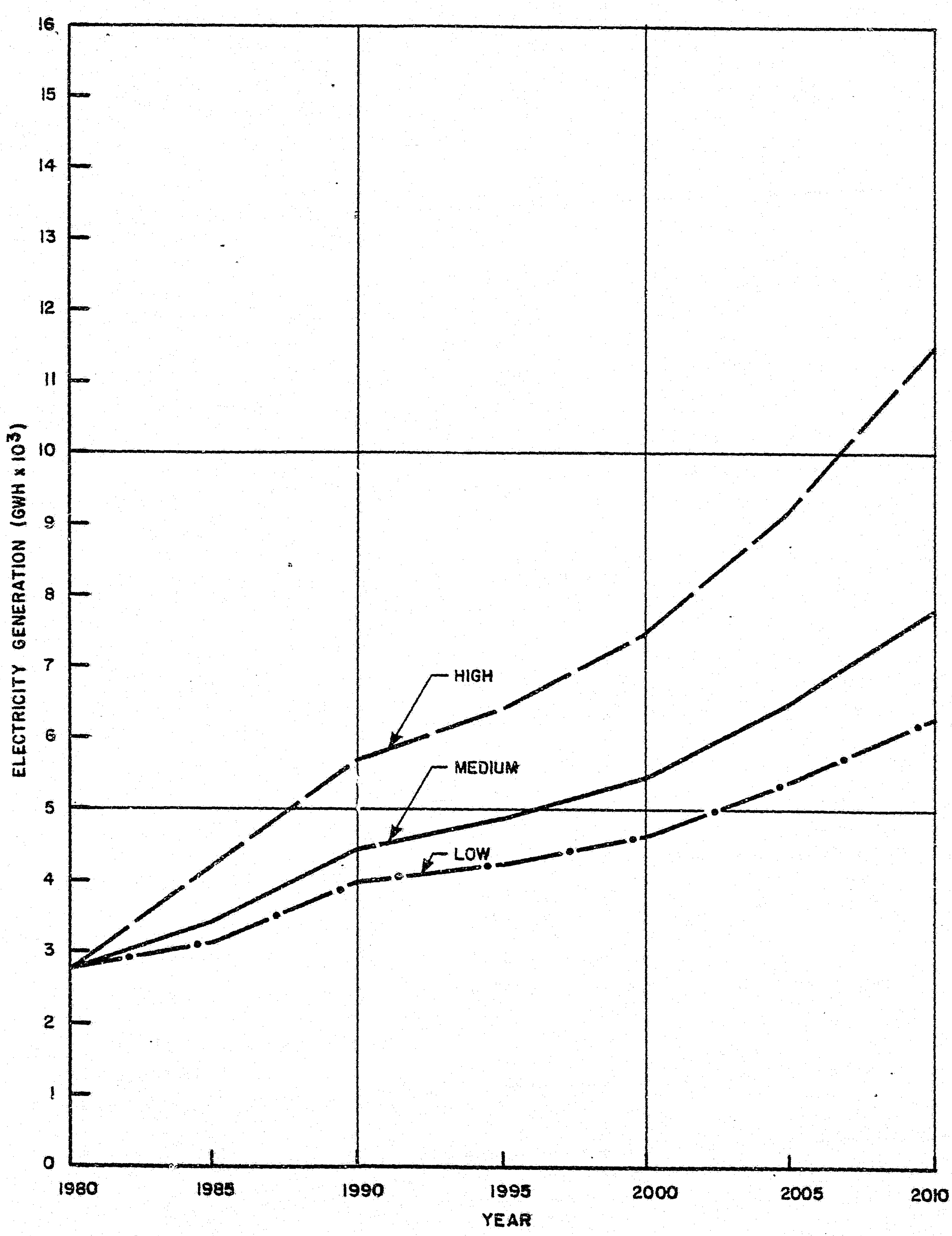
FIGURE B.62



ELECTRIC POWER FORECASTING PROCESS

FIGURE B.63

ACRES



DECEMBER 1981 BATTELLE LOAD AND
ENERGY FORECASTS USED FOR GENERATION PLANNING STUDIES

ACRES

FIGURE B.64

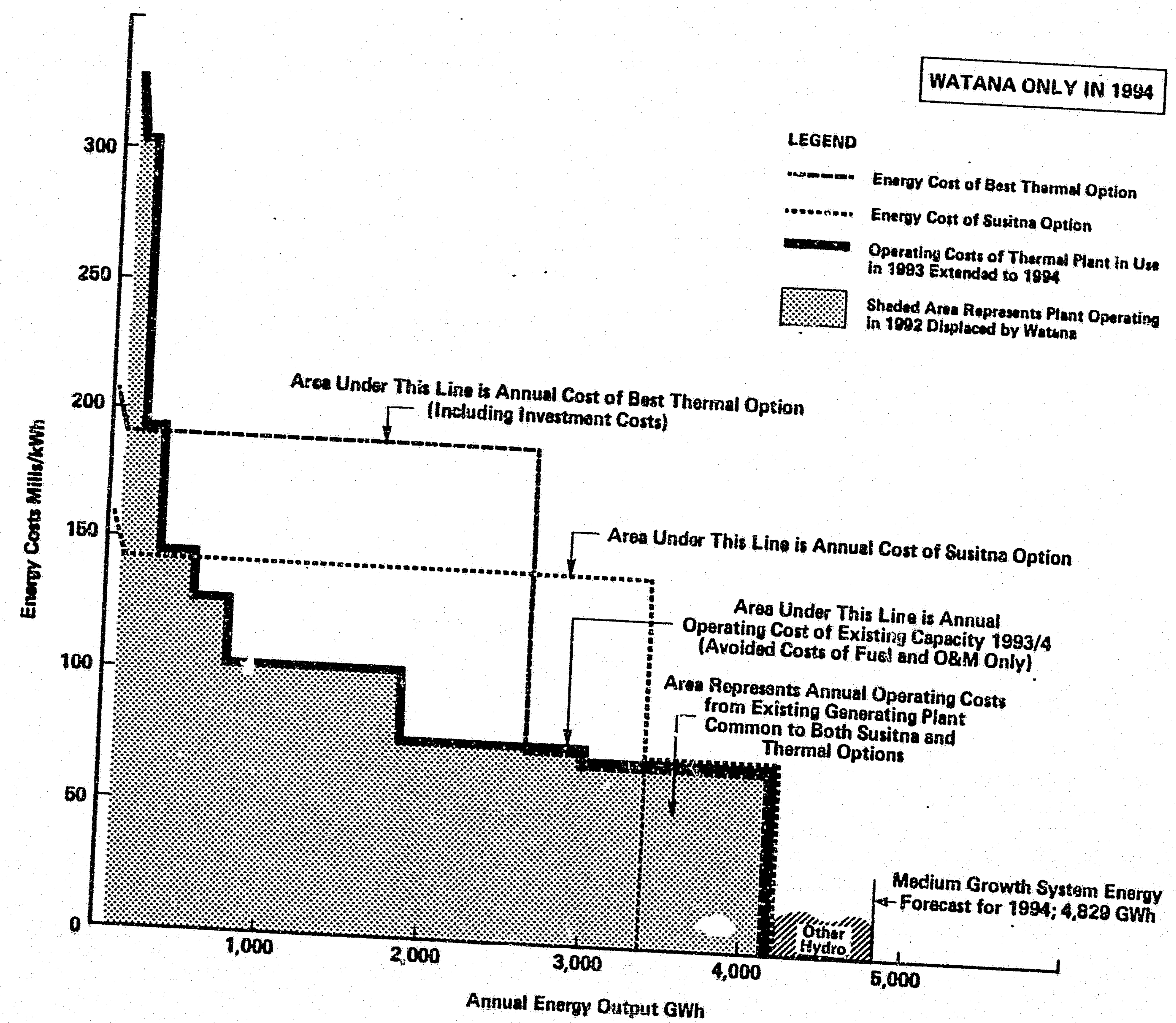


FIGURE B.65 – ENERGY PRICING COMPARISONS – 1994

ACRES

Rev. 1

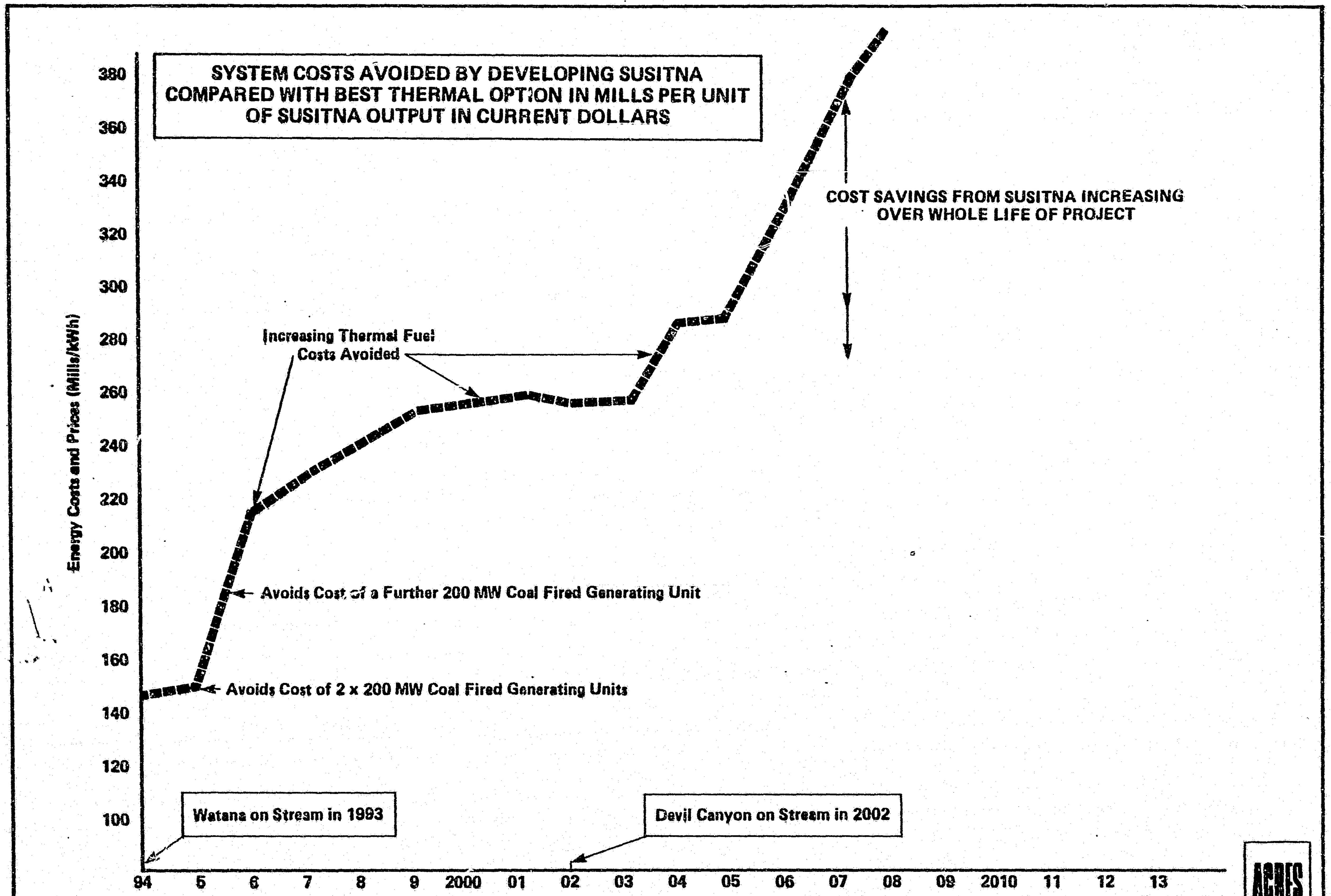


FIGURE B.66 — SYSTEM COSTS AVOIDED BY DEVELOPING SUSITNA