ALASKA POWER AUTHORITY
SUSITNA HYDROELECTRIC PROJECT

FEASIBILITY REPORT

VOLUME 1 - ENGINEERING AND ECONOMIC ASPECTS

FIRST DRAFT

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SUSITNA HYDROELECTRIC PROJECT

FEASIBILITY REPORT

PRELIMINARY OUTLINE

VOLUME 1 - ENGINEERING AND ECONOMIC ASPECTS

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9 - SELECTION OF WATANA GENERAL ARRANGEMENT

This section describes the evolution of the general arrangement of the Watana project, which, together with the Devil Canyon project, comprises the development plan selected as part of Section 8. This section also describes the site topography, geology, and seismicity of the Watana site relative to the design and arrangement of the various site facilities. The process by which reservoir operating levels and the installed generating capacity of the power facilities were established is also 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 obviously 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. A detailed description of the various project components is given in Section 12.

9.1 - Site Topography

The project site is located in a broad U-shaped valley at river mile 183, approximately 2-1/2 miles upstream of the confluence of Tsusena Creek with the Susitna River. The river at the site is relatively wide, although turbulent (Figure 7.11). On the right bank, the valley rises at an approximate slope of 2H:1V from river level at elevation 1450 for approximately 600 feet, then gradually flattens to a maximum elevation of 2350 between the Susitna River and Tsusena Creek. The left bank rises more steeply from the river for about 450 feet at a slope of 1.4H:1V, then flattens to 3H:1V or less to approximate elevation 2600.

9.2 - Site Geology

General

This section summarizes the geological and the geotechnical investigations conducted to date and the geologic conditions present at the Watana site. A detailed description of the geology and site investigations is presented in the 1980-1981 Geotechnical Report (1).

(a) Geologic Setting

The Watana site is located on the western side of a Tertiary age (2 to 70 m.y.b.p) intrusive body. The rock is primarily a gray to green medium grained crystalline igneous rock of diorite-quartz diorite composition. Associated with the pluton are andesitic dikes and volcanic flows, which are generally the fine grained equivalents of the intrusive rocks, and volcaniclastic sedimentary rocks.

The underlying rock is hard, fresh, and of good quality. It is slightly weathered at the surface and along joint surfaces. The overburden is generally thin on the valley walls, thickening to the north of the damsite, and consists of glacially derived silts, sands, and gravels. Shear, fracture, and alteration zones have been delineated highlighting two major structural features to be considered in the design of the dam. No evidence of major structural deformation or faulting has been found. Permafrost conditions exist on the north facing slopes with temperatures near the freezing point.

(b) Geological and Geotechnical Investigations

Surface and subsurface investigations for the site have been conducted by several organizations at different times. Preliminary reconnaissance work was done by the USBR in the 1950s. During the years of 1975 and 1978 the COE conducted site specific investigations to determine the suitability of the site. As part of the current study program, more detailed investigations were undertaken in 1980 and 1981 to establish the technical feasibility of the project. The investigations have included air reconnaissance, air photo interpretation, geologic mapping of rock and surficial material, drilling of both rock and overburden, including in-hole geophysical tests and seismic refraction surveys. Both insitu and laboratory tests have been performed to determine the engineering characteristic of soils and rocks. The location of drill holes and other investigations is shown in Figures 9.1 and 9.2.

Geologic mapping was concentrated in the immediate proposed damsite area between Tsusena and Deadman Creeks. All accessible areas were mapped for the rock and overburden exposures. The lithology or type of material, bedding, jointing, weathering, degree of consolidation, exposure size, and elevation were noted and plotted on maps for use in the interpretations.

Seismic refraction surveys were performed throughout the investigations on both banks, the river channel, the borrow areas, and the upper slopes of the damsite area to determine the bedrock depth and other significant features. A total of ____ feet of seismic refraction traverse were run at the Watana site. Information gathered was integrated and used with the other subsurface information for correlation and development of a three dimensional representation of site characteristics.

Diamond core and rotary drilling were performed in the foundations and abutments of the proposed project structures. A total of ___ feet was drilled in 28 holes with ___ feet of core recovered. In-hole permeability tests were conducted in each hole upon completion and geophysical logging and borehole photographs were performed in selected borings.

To monitor the ground water and ground temperature conditions at the site, piezometers and thermistor strings were installed in selected drill holes both in 1978 and 1980. A regular monitoring program for those instruments has been established to collect data.

A series of tests were performed on the rock recovered from coring to determine the engineering characteristics of the rock mass. The results of these investigations were used to develop the geologic picture of the damsite and the adjacent areas. Figures 9.3 and 9.4 present the rock outcrop map and the structural geology map of the site. The results of the laboratory rock tests are summarized in Table 9.1.

(c) Construction Material Investigations

Extensive investigations have been conducted both prior to and during the current studies to identify sufficient quantities of suitable materials for the construction of an embankment dam and for concrete aggregates. The investigation methods have included geologic mapping, auger and rotary drilling, excavation of test pits, seismic refraction surveys, and laboratory tests. A total of ___ rotary holes, ___ auger holes, ___ test pits, and ___ linear feet of seismic refraction surveys have been performed. The locations of the various potential sources of embankment material and concrete aggregates together with the locations of boreholes and test pits are shown in Figure 9.5.

(i) Rock Fill Material

Two sources for the rockfill material, designated Quarry A and Quarry B, have been identified. The rock in Quarry A is located on the south bank and primarily contains a relatively thin layer of andesite (an extensive igneous rock) overlying diorite. The diorite is generally hard, durable and fresh, and suitable for use as rockfill in the dam. Quarry B is an alternative source of rockfill, however, it is estimated that sufficient quantities of rock are available in Quarry A to meet the project requirements.

(ii) Impervious Core Material

Two sources have been identified for the impervious core material and are designated as Borrow Area D and Borrow Area H. Borrow Area D is located within 1.5 miles upstream of the damsite on the north bank. The upper few feet of material comprises tundra, topsoil, and boulders, and is underlain by glacial tills composed of dense gravelly silty sands with some clay. The tills range from 15 to 25 feet in thickness and overlie sandy gravelly clay. A composite grain size curve for these soils is presented in Figure 9.6. The material is well graded with natural water content at about the plastic limit. Figure 9.7 presents the results of Atterberg limits on the finer portion of the material. More than adequate quantities of material are present in Borrow Area D for the impervious fill requirements at Watana.

An alternative source of core material, designated Area H, is located approximately 5 to 7 miles downstream from the damsite on the south bank of the river in the Fog Lakes area. This area contains a relatively thick layer of till composed of silt, sand, and gravel with some cobbles. A composite grain size curve for the till material is presented in Figure 9.8 and the Atterberg limit test results in Figure 9.9. The material is quite similar to that in Borrow Area D except that the natural water content is higher. Sufficient quantities of impervious fill material are also available at this location if required.

(iii) Filter Material

Borrow Area E has been identified as a primary source of material for filter and transition zones of the embankment. This area is located at the confluence of the Tsusena Creek and the Susitna River approximately 2.5 miles downstream from the damsite. The area is covered by about 2 feet of organics and silt and is underlain by a few feet thick layer of silty sand to clean sand. Below that a thick layer of sandy and gravelly material exists. A composite grain-size distribution curve for Borrow Area E material is presented in Figure 9-10. Sufficient quantities are available in this borrow area to meet the project requirements for wilter materials. The material will have to be processed to meet the gradation requirements of specific zones. Additional alternate sources of material identified include Borrow Area C and Borrow Area F, at greater distances from the damsite. Also, some material from Borrow Area D and riverbed alluvium (as described later) is suitable with processing to be used as filter and/or transition material.

(iv) Gravels and Cobbles for Shells

Seismic refraction survey investigations were performed within ___ miles upstream and ___ miles downstream from the damsite in the Susitna River valley. These investigations confirmed that sufficient quantities of granual material are available for use in the supporting shell zones of the dam. In the upstream shell of the embankment, to meet design requirements, the material will require processing to remove fines and wastage of material 3/8 inch or smaller in size, and oversize material larger than 18 inches. Available data indicates that the grain size distribution of these materials will be similar to that from Borrow Area E, with probably a higher percentage of coarser material.

(v) Concrete Aggregate

The material available from Borrow Areas E, C, F, and the riverbed alluvium is suitable for use as coarse and fine aggregate for concrete. Processing will be required to produce desired gradations. The coarser particles are rounded and petrographic analyses have indicated the material to be of good quality. Sufficient quantities are available within the identified sources.

(d) Geologic Conditions

A summary of overburden and bedrock conditions is presented in the following paragraphs:

(i) Overburden

Overburden thickness is generally thin on the valley walls and thickens away from the proposed damsite to the north (Figure 9.3). On the lower slopes, the overburden consists primarily of talus. Above the break in slope where the topography becomes more gentle, glacial silts, sands, gravels, and boulders are encountered. Subsurface investigations have indicated the contact between the overburden and bedrock to be relatively unweathered.

The depth of the river alluvium beneath the proposed dam averages about 80 feet, up to a maximum of about 100 feet, and consists of sand, silt, coarse gravels, and boulders. Very little is known at this time about the denseness and character of this alluvium. A deep bedrock depression has been delineated on the north side of the river and is discussed under Section 9.13.

(ii) Bedrock Lithology

The Watana site is underlain by a serie of sedimentary, volcanic, and plutonic rocks. The damsite is primarily underlain by an intrusive dioritic body which varies in composition from granodiorite to quartzdiorite to diorite. The volcanic rocks (diorite and andesite) are generally finer grained equivalents of these intrusive rocks. The sedimentary rocks consist of tuffaceous siltstones and graywackes. The geologic map of the site is shown on Figure 9.3.

The quartz diorite is light gray and is found primarily upstream from the damsite axis. The texture is massive and the rock is hard, competent, and fresh except within the shear zones, which are discussed later. The diorite is a dark grayish green rock with massive appearance. It is hard, competent, and generally fresh. These two grades of diorite occur in alternating zones on the order of several hundred feet wide. Weathering is limited to a very thin zone on the exposed surface, and along the joints to ____ feet depth. These rocks have been intruded by mafic and felsic dikes which are generally only a few feet thick with tight contacts. These dikes generally trend parallel to major joint sets. The rock, downstream from these diorites, is a series of extrusive rocks ranging from rhyolite to andesite and basalt. Andesite porphyry is the more prominent of these rocks. The rock is a medium to dark gray to green and contains quartz diorite inclusions. The contact of the andesite with the underlying diorite is generally slightly weathered and fractured. On the north bank, the andesite-diorite contact at the downstream is coincident with the "Fingerbuster" shear zone.

A sequence of sedimentary rocks composed of essentially volcanic debris is exposed downstream from the damsite and is comprised of generally sound sandstones and siltstones. This sequence is overlain by andesite.

(iii) Bedrock Structures

- Joints

There are two major joint sets and two minor joint sets at the site (Figures 9.4 and 9.12). These joint sets are described in Table 9.2. Set I, which is the most prominent set strikes 320° (N40W) and dips 80° NE to vertical. This set is found throughout the damsite and parallels the general structural trend in the region. Joint Set I has a subset, which strikes 290° to 300° (N60 - 70W) with a dip of 75° NE. This subset is localized in the downstream area near where the diversion tunnel portals are planned. This subset also parallels the shear zones in the downstream area of the site. Set II trends northeast to east and dips vertically. This set is best developed in the upstream portion of the damsite area, although it is prominent in the downstream areas with a more easterly strike. No other structural features were found with orientations in this set. Sets III and IV are minor sets but can be locally strong. Set III forms numerous open joints on the cliff faces near the "Fingerbuster", and several shear zones parallel this orientation. Set IV appears to result from stress relief from glacial unloading and/or valley erosion.

The average spacing of Joint Sets I and II is 1 to 2 feet and 6 to 12 inches, respectively. The spacing of Joint Sets III and IV is quite variable and can range from a few inches to several feet.

- Shears and Fracture Zones

Several shears, fracture zones, and alteration zones are present at the site. For the most part, these are small and discontinuous. During mapping, all zones greater than 10 feet in width were mapped and delineated on the geologic may (Figure 9.4).

Shears are defined as having breccia, gouge, and/or slickenslides indicating relative movement and are found in two forms. The first type is found only in the diorite and are characterized by breccia of sheared rock that has been healed by a matrix or very fine grained andesite/diorite. The contacts, although irregular, are tight and unfractured. The zones were found to have high RQDs and to be fresh and hard. The second type is common to all the rock types and consists of breccia and/or gouge of fine grained rock types and consists of breccia and/or gouge of fine grained rock material in a silt/clay matrix. These are soft, friable, and often have secondary mineralization of carbonate and chlorite showing slickensides. These are generally less than 1 foot wide.

Fracture zones are also common to all rock types ranging from a 6-inch to 30-foot wide (generally less than 10 feet) zone of closely spaced joints that are often iron oxide stained or carbonate coated. Where exposed, the zones tend to form topographic lows.

In the alteration zones, the felspars and mafic minerals of the rock have been chemically altered by hydrothermal solutions to clay and chlorite. The degree of alteration encountered is highly variable across the site. These zones are rarely seen in outcrop as they are easily eroded into gullies but were encountered to some degree in all the boreholes. The transition between fresh and altered rock is gradational and the zones may range to 20 feet thick although are usually less than 5 feet. Core recoveries are generally very good and the rock quality is dependent on the degree of alteration.

(iv) Significant Structural Features

The Watana site has several significant geologic features consisting of broad areas of the shears, fractures and alteration zones described previously.

The two most prominent areas have been named "The Fins" and the "Fingerbuster". "The Fins" is located on the north bank of the river upstream from the diversion tunnel intake. It is approximately a 400 foot wide area characterized by three major northwest trending zones of shearing and alteration that have eroded into steep gullies. These alteration zones are separated by intact rock bands (ribs) 5 to 50 feet wide. The 20-foot wide upstream zone of the series coincides with the diorite/andesite porphry contact. The other two zones, approximately 55 and 30 feet wide, are filled with severely altered talus. This area is characterized by a 300 - 310° orientation (N50 to 60W) and near vertical dips of its component shears and by low seismic refraction velocity of the rock. The extension of the zone is extrapolated by topographic and seismic lows northwest to a sheared/altered outcrop on Tsusena Creek.

The "Fingerbuster" is located downstream from the damsite and is exposed in a 40 foot wide deep talus filled gully along the andesite porphyry/diorite contact. The rock is severely weathered with closely spaced joints trending parallel to Set I (330°) and Set III (0°). Slickensides indicate vertical displacement. The extension of this zone to the south is based on a strong north-south topographic lineament, although no outcrops are exposed. Because of the lack of exposure and definable continuity of this feature, its location orientation is extrapolated.

A prominent alteration zone was encountered in BH-12 on the south bank. The hole encountered approximately 200 feet of hydrothermally altered rock. Although core recovery in thin boring was very good, the quality of rock was relatively poor and a zone of significant artesian pressure was encountered. The south eastern extension of this zone passes under the core of the dam.

(e) Ground Water Conditions

The ground water regime in the bedrock is confined to movement along fractures and joints. Measured water levels have ranged from ____ to ____ feet below surface. It is assumed that the ground water in the non-permafrost areas on the north side of the river is a subdued replica of the topography with the gradient towards the Susitna River and its tributaries. Artesian conditions are present in isolated fracture isolation zones.

(f) Permafrost Conditions

Permafrost conditions exist on the north facing slopes (left bank) of the damsite area. Measurements in the borings indicate that it penetrates to a depth of ____ feet and show marginal temperatures within 1°C of freezing. Only sporadic areas of permafrost have been encountered on the right bank.

(g) Reservoir Geology

The topography of the Watana Reservoir and adjacent slopes is characterized by a narrow V-shaped stream-cut valley superimposed on broad shapes U-shaped glacial valley. Overburden masks much of the bedrock especially in the lower and uppermost reaches of the reservoir. Figure 7.11 shows the general Watana reservoir geology.

The lower portions of the Watana reservoir are predominantly covered by a veneer of glacial till with scattered outwash deposits. On the south side of the Susitna River, the Fog Lakes area is characterized by a fluted ground moraine surface. Upstream in the Watana Creek area, a broad flat plain is mantled with glacial till and semi-consolidated Tertiary sediments. These are predominantly stratified, poorly graded, fine sands and silts with some clays. The river valleys contain significant amounts of alluvial deposits and reworked outwash. Ice disintegration features such as Kanes and eskers have been observed in the river valley.

A non-conformable contact between argillite and the diorite pluton in the damsite area was mapped approximately three miles upstream of the damsite. Semi- consolidated, Tertiary age sedimentary rocks, and volcanics of Triassic age are present just downstream from the confluence of Watana Creek and the Susitna Rvier. These volcanics consist of metabasalt flows with thin interbeds of metachert, argillite, marble, and metavolcaniclastic rocks. Metamorphosed from Watana Creek to Jay Creek. The rocks between Jay Creek and Oshetna Creek are metamorphic amphibolite and minor amounts of greenschist and foliated diorite.

The main structural feature of the Watana Reservoir is the Talkeetna Thrust Fault which trends northeast - southwest. The Talkeetna Thrust Fault crosses the Susitna River approximately eight miles upstream from the damsite. This fault has been studied in detail as part of the seismic studies, and has been determined to be inactive in relation to the project design.

9.3 - Geotechnical Design Considerations

This section deals with the geotechnical aspects of design of the dam and other major structures at the Watana site.

(a) Main Dam Excavation and Foundation Treatment

As discussed previously, the riverbed alluvium ranges up to approximately 100 feet in depth. The character of the this material is difficult to define, however, its stability during a strong earthquake event is questionable. Considering the nature of the material, and the height of the dam, the riverbed material will be removed entirely within the limits of the dam. the overburden material on the abutments is relatively thin, except for gullies and pockets. Most of this material is frozen and will become unstable when thawed and is therefore unsuitable for the dam foundation. Accordingly, the overburden on the abutments will also be required. Details of foundation treatment are presented in Section 12.

The presence of numerous shear zones, containing gouge material, indicates the need to remove all weathered rock under the impervious core and upstream and downstream filter zones. Excavation will include shaping of valley walls along the abutments to provide a proper contat surface in accordance with good modern design practice. Excavation under the outer shells will include removal of loose rock blocks, extensively weathered rock and local reshaping as necessary. The strength of the rock foundation is otherwise adequate to support the embankment and associated reservoir loads.

The two major geologic structures at the site, "The Fins" on the upstream side and the "Fingerbuster" on the downstream side, have a major influence on the overall project layout, however, they do not directly affect the dam at its proposed location.

Extensive permafrost is present on the south bank (as deep as several hundred feet) and sporadic permafrost has been noted on the north bank. This permafrost is within 1°C of freezing and is protected in most part by the thick tundra and heavy vegetation. During the foundation excavation, the loss of insulating tundra may cause thawing of overburden and may result in unstable slopes and/or mud flows. Therefore, proper care and planning will be required during removal of this material.

(b) Control of Underseepage and Uplift

A grout curtain will be provided under the core of the dam and extending several hundred feet into the abutment beyond the dam. The initial phases of the grout curtain will be exploratory in nature, in order to identify areas in the dam foundation rock that require more extensive treatment and provide detailed information of the foundation. A series of drain holes will be drilled downstream from the grout curtain to form a continuous drainage curtain. These holes will drain into underground grouting/drainage galleries, which will permit monitoring of seepages and water pressures and access for necessary remedial work if required at a later date. Since the rock mass is permafrost-affected, ground thawing will be required prior to grouting. Consolidation grouting is all planned under the core and the filters to provide a good contact surface free from open joints and fractures.

(c) Cofferdams and Dewatering

Because of the pervious nature of the thick riverbed desposits and the relatively high pool behind the upstream cofferdan, major dewatering operations will be required during the foundation excavation and until the dam construction reaches above the diversion stage pool level. A slurry trench cutoff is currently proposed beneath the upstream cofferdam to control water flows during diversion. Further exploration is necessary in the riverbed to better define the extent and condition of the alluvial materials at the cofferdam site prior to construction.

(d) Underground Structures

The rock conditions at the Watana site are suitable for the construction of tunnels and underground caverns. From the geological and geotechnical viewpoint, the location and the orientation of these structures is influenced by the orientation and location of rock discontinuities. Permafrost conditions will not have any major adverse impact except where thawing may be required for grouting.

The RQD values indicate that 85 percent of the rock is of a good to excellent category. The remaining 15 percent represents poor quality rock associated with rock discontinuities. The major joint sets at the Watana site are oriented at N40W (Set I) and N45E (Set II). Other four joint sets are minor. The major shear and fracture zones also parallel these general trends. The most favorable orientations for the tunnels and the large underground caverns are those with their long axes perpendicular to the major joint sets. These factors have been a major factor in selection of the alignments of the tunnels and major caverns to achieve maximum stability and minimum support requirement.

Although little is known at this time about the insitu stress regime at the site, the general tectonic stress regime within the region is in a compression mode. The unconfined compressive strength of the rock ranges from ___ to ____ and suggests that overstressing problems such as spalling or slabing are not likely. Conventional rock bolt support using 1 inch diameter bolts is generally considered adequate in most areas with spans less than 40 feet. For larger spans and in areas of poor quality rock, the support requirements have been determined on a case-by-case basis. In the case of large span openings, intersection of nearby vertical and subhorizontal joints can create unstable blocks in the crown. Allowances have been made for the use of support measures such as shotcrete, welded wire fabric, and concrete lining in areas of potentially poor rock quality and water carrying tunnels under high head (such as penstocks).

Although the rock mass by itself is fairly impervious, intersection of rock discontinuities may cause ground water problems during construction and act as path of seepage and high pore pressures during operation. Provisions have, therefore, been made for consolidation/ring grouting and suitably placed drain holes to reduce the risk of a build up of high pore pressures.

Tunnel excavation can be performed using conventional drill and blast techniques or high production mechanical excavations. Sufficient information is not available at this time to make this decision, and for feasibility assessment purposes, conventional drill and blast methods have been assumed. The excavation of powerhouse caverns will be performed by drill and blast using a primary heading, side slash and bench excavations approach.

The spacing between long tunnels has been set at 2.5 times the diameter of the largest tunnel. The spacing between the major caverns has been set such that a pillar thickness of 1.5 times the span of the larger cavern is maintained.

(e) Stability of Soil and Rock Slopes

In most areas the excavation slopes will be in the rock. The slopes in the overburden, where necessary, have been based on the nature of soil, ground water table, and the height of the slope. In general, slopes in overburden will not be steeper than 2H:1V below the water table and 1.5:1V above the water table. A bench of adequate width will be provided at the overburden-rock contact to accommodate any local slumping or slope failure and to intercept and dispose of ground/seepage water. Flatter slopes may be required where frozen ground may be become unstable because of high pore pressures during thawing.

The slopes in rock are controlled by the joint dips and orientations. Since major joint set dips are almost vertical, 1H:10V slopes are considered reasonable up to 40 feet in height. Where the height of slope exceeds 40 feet, a minimum of 10 foot wide bench have been provided every 40 feet depth to facilitate construction and to provide access for future maintenance. These berms will also intercept falling loose rock pieces and surface/ground water drainage. Locally, rock bolting or similar support techniques and drain holes have been provided in appropriate areas to maintain stable rock slopes.

Excavation of tunnel portals will be accomplished by liberal use of pattern rock bolting and some provision for concrete/shotcrete to reduce the risk of unstable slopes. Special details have been incorporated in areas where slopes intersect or cross larger shear zones or otherwise unstable rock.

(f) Use of Excavated Rock in Dam Construction

Since most of the rock excavation will be within the diorite and andesite rocks, the quality of rock will be acceptable for use in the rockfill portions of the dam. The exception to this could be excavation in poor quality or weathered rock which will result in unacceptable fill. The use of the rockfill in the dam will be be limited to portions of the downstream shell, and in zones of rip rap material. Proper quality control will be exercised in selecting this material.

(g) Relict Channel

A deep bedrock depression exists on the north bank of the river extending from about 2,500 feet west of Deadman Creek northwest toward Tsusena Creek. The depth to bedrock is as much as 400 feet below the surface and the reservoir level. The overburden consists of several sequences of glacial deposits, lake sediments, and alluvium varying in thickness and character both laterally and with depth. Some of these granular deposits exhibit high permeability, and ice inclusions were noted at a depth of several hundred feet suggesting the possibility of permafrost. The ground water surface has not been well defined and a perched water table has been encountered in at least one boring indicating artesian pressure, and is also evidenced by the presence of several surface lakes. With the proposed range of reservoir levels, these overburden deposits will become saturated. A bedrock contour map of the Relict Channel area is presented in Figure 9.13.

A saddle dam of relatively low height is planned across the topographic low of this Relict Channel. Details of the potential design problems to be dealt with in the Relict Channel and the proposed methods of treatment are discussed further in Section 12. Additional investigation will be necessary to properly characterize the subsurface condition and the final details of foundation treatment in the area prior to construction.

9.4 - Seismic Considerations

The seismicity of the Susitna Basin and the sources of earthquakes are discussed in Section 7 of this report. This section presents the implications of the seismicity on the design of the Watana project.

(a) Seismic Design Approach

For earthquake engineering and design considerations, the project structures have been classified as either critical structures or non-critical structures. Critical structures include the dam and similar major structures whose failure may result in sudden and uncontrolled release of large volumes of water which may endanger property and lives downstream. The non-critical structures are those structures whose failure can be assessed as an economic or financial loss to the project in terms of lost revenue, repair, and/or replacement cost. Critical structures will be designed to safely withstand the effect of the "Safety Evaluation Earthquake" (SEE) for the site. No significant damage to these structures will be accepted under these conditions. The design of non-critical structures for earthquake conditions is undertaken using conventional Uniform Building Code recommendations.

For design of critical structures the effective acceleration for the SEE has been determined as 0.8 x actual SEE acceleration. In the case of the earthfill structures, designs are based on analyses using a projected time history for the selected earthquake event. For other structures a correspondingly scaled response spectrum is used. The selected SEE for Watana was based on a consideration of two of the most severe events which might occur. The first of these is the "terrain" or "detection level" earthquake which has been characterized as follows:

- Magnitude: 6-1/4 to 6-1/2

- Location: Approximately 3 km from structure

- Maximum Acceleration: Mean 0.55 g to 0.60 g
  84th percentile 0.70 g

- Peak Spectral Acceleration: Mean 1.37 g to 1.50 g
  84th percentile 1.77 g

The duration of this event is relatively short and the time history approach to design was not developed. The response spectra for this event are shown on Figure 10.7A. The effective peak acceleration for design of structures is then:

- Design a max = 0.8 x 0.70 g = 0.56 g
  (Sa) max = 0.56 g x 2.5 = 1.40 g

The second earthquake source to be considered for design at Watana is the Benioff Zone.

(b) Safety Evaluation Earthquake for Watana

Although the "terrain" earthquake would result in more severe ground motions, the duration of these motions is relatively short and the likelihood of occurrence of such an event is extremely small. A more likely source of strong ground shaking at the Watana site is the Benioff Zone. The estimated mean peak response spectrum for the SEE for this event is presented in Figure 7.14, along with the 84th percentile response spectrum. A maximum horizontal acceleration level for the 84th percentile response spectrum for the Benioff event is approximately _____.

The design of the Watana Dam has been based in the projected time history for this event as shown in Figure ___, and as discussed in Section 12.

9.5 - Selection of Reservoir Levels

This section describes the approach used and the results of the evaluations made in the determintion of optimum Watana reservoir level. The selected elevation of the Watana dam crest is based on considerations of the value of the hydroelectric energy produced from the associated reservoir, and geotechnical constraints on reservoir levels. 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 feet. The relative value of energy produced for each of these three dam elevations was then determined by means of the OGP generation planning model as discussed in Section 6, to determine the long term present worth cost of meeting the Railbelt system energy demand. Finally the physical constraints imposed on dam height and reservoir elevation by geotechnical considerations were reviewed and incorporated into the crest elevation selection process.

(a) 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 mult-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 (see Section 9.11). Minimum flow requirements were established at both project sites based on mechanical plant-related restrictions and downstream fisheries considerations.

As discussed in Section 9.6, 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 650 MW at Watana in 1993, increasing to 1020 MW at Watana in 2007, with an additional 600 MW at Devil Canyon coming on line in the year 2002. Yearly system demand and monthly and daily load patterns within the Railbelt over a 29 year period were based on forecasts developed as described in Section 5 and 6. 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 OGP V model.

As discussed in Section 6, these present worth costs are based on economic parameters, not of inflation. The construction cost estimates used in the OGP V 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 in the process of development of the estimates presented in Section 16. These refinements have no significant impact on the reservoir level selection. The basis of assumed costs for construction and operation of alternative energy generation facilities is also discussed in Section 6.

(b) Optimization

Optimization of the Watana reservoir level was based on an evaluation of three dam crest elevations of 2240, 2190, and 2140 feet. These crest elevations apply to the central portion of the embankment with appropriate allowances for freeboard and seismic slumping, 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 9.3, together with corresponding project construction costs.

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

The results of these evaluations are shown in Table 9.4, and plots showing the variation of the long term present worth with dam crest elevation are shown in Figure 9.17. This figure indicates that on the basis of the assumptions used, the minimum long term present worth of production costs occurs at a Watana dam crest elevation ranging from approximately 2160 to 2200 feet (reservoir levels 2140 to 2180). A higher dam crest will still result in a development which has an overall net economic benefit relative to displaced 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 alternative energy sources. Hence, the long-term present worth of the overall system increases. Conversely, as the height of the dam is lowered, and thus Watana produces less energy, the unit cost of the energy produced by alternative generation sources to replace the lost Susitna energy, is more expensive than Susitna energy. In thise case also, the long-term present worth again increases.

(c) Conclusions

It is important to clearly establish the overall objective of setting the Watana reservoir level. An objective which is to minimize the long term present worth of energy cost will lead to selection of a lower reservoir level than an objective which is to maximize the amount of energy which can be obtained from the available resource, while still doing so economically and within accepted technical and environmental constraints.

The three values of long term present worth developed by the OGP V computer runs defined a relationship between long term present worth cost and Watana dam height which is relatively insensitive to dam height. There is an indication that a small difference in system present worth occurs as the Watana dam crest is raised or lowered ovesr the range considered. However these differences are of the same order as the inaccuracies which are inherent in capital cost estimates for the development of such major generation facilities within the Railbelt. Little value would thus be gained from analyzing intermediate dam heights to further define the curve. The insensitivity is highlighted by the graph of present worth against dam height in Figure 9.17. This figure shows these slight variations in context within the total long term present worth cost of the system.

Thus, from an economic standpoint, the optimum crest elevation could be considered as varying over a range of elevations of as much as 50 to 100 feet. The governing factors is establishing the upper limit of dam height were consequently physical and geotechnical considerations, allowing the objective of maximizing the economic use of the Susitna resource still to be satisfied.

The normal maximum operating level of the reservoir was therefore set at elevation 2185 feet. At this level, for up to the 1:10,000 year flood occurrence, there will be no danger of over-topping the low lying portion of the relict channel on the right side of the river. In the unlikely event of floods of greater severity, a freeboard dike in the low area of up to 10 feet in height has been incorporated in the design. 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.

9.6 - 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, 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.

(a) 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 operation computer simulation program based on the 32 years of hydrological record developed as described in Section 7 (see Plate 22).

- An assessment was made of the alternative thermal energy required to meet the predicted load forecast, using a computer simulation of economic load dispatch from available plant in the firm year. This determined the optimum scheduling and capacity of new thermal plant required to meet the minimum Loss-of-Load Probability (LOLP) criterion for system security.

- 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, hydro energy in excess of system demand, displaces thermal energy (from coal, gas turbine, combined cycle, or diesel plant).

- 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 hydro energy is used to supply base load requirements with thermal energy at peak demand (Option 2). The actual choice is based on made on economic load dispatch criteria.

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

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

(b) Total Installed Capacity

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

Demand Year	Capacity (MW) Option 1	Capacity (MW) Option 2
1993	801	801
1995	839	839
2000	862	742
2002 (Incl. Devil Canyon)	660	655
2005 (Incl. Devil Canyon)	750	740
2010 (Incl. Devil Canyon)	908	900

On the basis of this evaluation, the ultimate power generation capability at Watana was selected as 1000 MW for preliminary design purposes, to allow a margin for hydro spinning reserve and standby for forced outage. This installation also provides a low cost margin in the event than an accelerated growth of demand occurs.

(c) 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, since, for example, Francis turbines will not operate effectively at less than about 50 percent full output. Growth of the actual load demand is also a significant factor, since the unit installation may be phased to match the actual load growth.

The number of units and their individual ratings were determined by the requirement to deliver the design peak capacity in the critical demand month of December, at minimum December reservoir level, with turbine wicket gates fully open. In addition, unit selection was based on consideration of the following:

- Rate of load growth with time;
- Load following capability at part station operation;
- Efficiency variation with load and head;
- Minimum acceptable load on each machine;
- Minimum downstream compensation flow;
- Standby capacity and spinning reserve; and
- Sensitivity to change in forecasted load growth.

An examination was made of the economioc impact on power plant production costs for various combinations of unit numbers and rated capacity, which would provide the selected capability of 1000 MW and satisfy the considerations outlined above. As discussed above, for any given installed capacity, plant efficiency increases as the number of units increases. This is illustrated in Figure 9.18. The assumed capitalized value of the resulting additional annual energy used for this evaluation was 1000 mills per kWh, based on economic parameters developed in previously described system studies. Variations in unit numbers 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)
4	250	-	-
6	170	40	31
8	125	50	58

It is apparent from this analysis that a six-unit scheme is the most economic alternative. This scheme also offers a high degree of flexibility and security of operation compared to the four-unit alternative, as well as advantages if unit installation is required to be phased to match actual load growth. The net economic benefits of the six-unit scheme are greater than those of the eight-unit scheme, while at the same time, no significant operational or scheduling advantages are associated with the eight-unit scheme. Accordingly, a scheme incorporating six units each with a rated capacity of 170 MW, for a total of 1020 MW, has been adopted for all alternatives.

For project design and cost estimating purposes in the current study, the installed capacity of 1020 MW has been assumed. From generation planning and financial analyses, certain advantages may be gained from staging the installation of generating equipment over a somewhat longer period. These aspects will be addressed further during detailed design of the project.

The power facilities and associated equipment are described in detail in Section 12.

9.7 - Selection of Spillway Capacity

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 overtopping 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 probable maximum flood without endangering the main dam or ancillary structures, in a manner which will avoid injury or loss of life, or damage to the project itself. 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 of the project if the spillway flood is exceeded, compared to the costs of the structures required to safely discharge the flood. A list of spillway design flood frequencies and magnitudes for several major projects is presented below.

Spillway Design Flood Frequencies and Magnitudes

Project	Spillway Design Flood Frequency	Spillway Design Flood Peak Inflow (cfs)	Basin PMF (cfs)	Spillway Capacity After Routing (cfs) Design*
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	2,195,000
Sayano, USSR	1:10,000	480,000	N/A	680,000

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

On the basis of the foregoing, a spillway design flood with a return frequency of 1:10,000 years was selected for Watana.

The flood frequency analysis undertaken as described in Section 7.2 indicates the following values:

Flood	Frequency	Inflow Peak
Probable Maximum	--	326,000 cfs
Spillway Design	1:10,000	156,000 cfs (0.47 PMF)

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

9.8 - Main Dam Alternatives

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

(a) Comparison of Embankment and Concrete Type Dams

The selection between an embankment type or a concrete type dam is usually made based on the configuration of the valley, the condition of the foundation rock and depth of the overburden, and the relative availability of construction materials. Previous studies by the COE envisaged an embankment dam at Watana. Initial studies as part of these current evaluation included comparison of a earthfill dam with a concrete arch dam at the site. An arrangement for a concrete arch dam alternative at Watana is presented in Plate 11. The results of this analysis indicated that the cost of the embankment dam was somewhat lower than the arch dam, based on the use of concrete costs significantly lower than comparable costs used for Devil Canyon. This preliminary evaluation did not indicate any significant advantages for the concrete arch relative to the arrangement of other structures, or the construction schedule relative to the use of a concrete arch.

Based on the overall cost differences described 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.

(b) Selection of Dam Type

The development of the design of the main dam, together with a description of the various features of the dam, is given in Section 12. The dam is, of course, the central and most costly component of the project, and a brief discussion of the development of the finally selected design, together with some of the factors which influenced development of the general arrangement are presented in this section.

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 impervious core material;

- The requirement that the dam be capable of withstanding the effects of a significant earthquake shock, 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 exploration program undertaken during 1980 and 1981 indicated that adequate quantities of materials suitable for dam construction were located within reasonable distance from the site. The locations of potential borrow materials for the dam are shown on Figure 9.6. The well graded silty sand material from Borrow Area D is considered the most promising source of impervious fill. Compaction tests indicate a natural moisture constant slightly on the wet side of optimum moisture content, so that control of moisture content will be critical in achieving a dense impervious core with high shear strength.

Potential sources for the upstream and downstream shells included either river gravel from borrow areas along the Susitna River, or compacted rock fill from structural excavation of quarries.

The main dam will consist of a compacted impervious core protected by fine and coarse filter and transition zones on both the upstream and downstream slopes of the core. The upstream and downstream outer supporting fill zones will comprise relatively free draining materials such as compacted gravel or rockfill, providing stability to the overall embankment structure. The location and inclination of the impervious 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.

The advantages and disadvantages of these two alternatives are discussed in Section 12. A central vertical core was chosen for the embankment based on a review of precedent design and the nature of the available potential impervious material.

In order to evaluate the relative sensitivity of the project arrangement to changes in exterior dam slopes, two alternatives were used in the preliminary review:

- 2.4H: 1V upstream and 2H:1V downstream
- 2.25H: 1V upstream and 2H:1V downstream

As part of the intermediate 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 Plate 12. The results of this comparison are described below:

Alternative	Total Volume (millions c.y.)
1	69.2
2	71.7
3	69.3
4	71.9

During the intermediate review, the upstream slope of the dam was flattened 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 shock 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 static and dynamic design analyses being undertaken at the same time as the general arrangement studies. This section was used for the final review of alternative schemes. Further refinements to the design were subsequently incorporated in the final design presented in Section 12, but these did not influence the selection of the final scheme.

9.9 - Diversion Scheme Alternatives

The topography of 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 configuration of the river in the vicinity of the site favors location of the diversion tunnel or tunnels on the right bank, since the tunnel length for a comparable scheme on the left bank could 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. Notwithstanding these considerations, the selection process for establishing the final general arrangement included examination of tunnel locations on both banks.

(a) 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. These last two considerations are usually evaluated as part of an economic risk analysis in which the cost of the diversion scheme, and the risks involved in exceeding the capabilities of the scheme. This type of analysis will be undertaken as part of the detailed design phase of the project, but for the purposes of the feasibility analysis, design criteria and experience from other projects similar in scope and nature have been used.

At Watana, damage to the partially completed project would 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 to operate for 4 or 5 years until the dam was completed sufficiently 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 over the critical 5 year construction period. The equivalent inflow for the design flood together with average flow characteristics of the river significant to diversion are presented below:

Average annual flow	7,860 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

(b) Cofferdams

The character and considerable depth of riverbed alluvium at both cofferdam sites indicate that embankment type cofferdam structures would be the only technically and economically feasible alternative at Watana. For the purposes of establishing the overall general arrangement of the project and for subsequent diversion optimization studies, the upstream cofferdam section adopted comprised an initial closure section approximately 20 feet high constructed in the wet, with a zoned embankment constructed in the dry. The downstream cofferdam comprises a closure dam structure approximately 30 feet high placed in the wet. Control of underseepage through the relatively pervious underlying alluvium material will be achieved by means of a soil/ bentonite slurry wall.

The selected cofferdam sections are described in more detail in Section 12.

(c) Diversion Tunnels

A basic consideration in evaluation of any diversion tunnel scheme is an examination of the advantages and disadvantages of concrete-lined tunnels compared to unlined tunnels. 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 feet per second have been used in several projects. For unlined tunnels, maximum design velocities ranging from 10 fps in good quality rock to 4 fps in less competent material are typical. Using a maximum permissible velocity of 10 fps, four unlined tunnels each with an equivalent diameter of 50 feet would be required to pass the design flow. Alternatively, a design velocity of 50 fps would theoretically permit the use of one concrete-lined tunnel with an equivalent finished diameter of 44 feet. The unlined tunnels would require 4.5 times as much excavation as the lined alternative, together with at least four times as much tunnel support cost. This would only be partially offset by the cost of the concrete lining. Apart from cost, the most important single factor relates to the security and reliability of the diversion scheme. The tunnels will undoubtedly traverse numerous unfavorable geological conditions and structures, as yet undefined, during construction. 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. These considerations, together with cost and the somewhat questionable feasibility of a tunnel with a diameter approaching 50 feet in this type of rock, are considered sufficient to eliminate consideration of unlined tunnels for the diversion scheme.

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.

Pressure tunnels are designed to flow full and accordingly must withstand internal pressure. The most widely used type of pressure tunnel for diversion has the crown of the outlet portal submerged during all flow conditions.

(d) Emergency Release Facilities

While not an integral part of the diversion scheme itself, the emergency release facilities greately 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, in accordance with current design practice, some form of low level release facility was required to permit lowering of the reservoir in the event of an extreme emergency. Since the primary discharge facilities will be located near the crest of the dam, they would be ineffective if the reservoir level had to be reduced below approximate elevation 1950. The most economical alternative available would involve converting an existing diversion tunnel to permanent use as a low level outlet facility. Since it obviously would be necessary to maintain the diversion scheme in effective service during construction of the low level outlet works, two or more tunnels would be required if this alternative was adopted. The use of two diversion tunnels, while not contributing to the overall economy of the project, provides an additional measure of security to the diversion scheme in case of the loss of service of one tunnel during an emergency. The use of two tunnels also provides greater flexibility in construction scheduling, particularly since concrete-lined tunnels are required. Additionally, potential problems with stability of two smaller openings are likely to be less severe than for the larger spans associated with a single tunnel.

If operation of the emergency low level release facilities is required, it will extend over a considerable period of time. Discharge of the facilities at the heads required could result in serious erosion downstream. This requirement necessitated some form of energy dissipation prior to returning the reservoir water to the river. Given the space restrictions imposed by the size of the diversion tunnel, it was decided to utilize a double expansion system with concrete plugs within the tunnel. The operation of the expansion chamber is described in Section 12. The use of this arrangement requires that the chamber be located above tailwater to prevent cavitation in the area of the emerging jets from the downstream plug. The implications of this restraint require that if a diversion tunnel is to be used as part of the emergency low level release facilities, it must act as a free flow tunnel.

(e) 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 type and diameter.

Capital costs were developed for three heights for an 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 was adopted.

Capital costs for the associated 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 and outlet gate structures and associated gates was determined not to vary significantly with tunnel diameter and was excluded from the analysis.

A right bank configuration was selected and the corresponding tunnel length in all cases was assumed to be 4,700 feet.

Curves of headwater elevation versus tunnel diameter for the various tunnel alternatives with submerged and free outlets are presented in Figure 9.13. The relationship between capital cost and crest elevation for the upstream cofferdam is shown in Figure 9.14. The capital cost for various tunnel diameters with free and submerged outlets is given in Figure 9.15.

The results of the optimization study are presented in Figure 9.16, and indicate the following optimum solutions for each alternative.

Type of Tunnel	Diameter (ft)	Cofferdam Elevation (ft)	Total Cost ($)
2-Pressure tunnels	30	1595	66,000,000
2-Free flow tunnels	32.5	1570	68,000,000
2-Free flow tunnels	35	1545	69,000,000

The foregoing indicates that a relatively small cost differential (4 to 5 percent) separates the various alternatives in range of tunnel diameter from 30 to 35 feet.

(f) Selected Diversion Scheme

Although a scheme incorporating two 30 foot diameter pressure tunnels with submerged outlets is marginally the most economical solution as discussed in (d), at least one tunnel must have a free outlet if it is to be converted into a low level outlet tunnel.

An important consideration at this point is 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 riverbed level, and diversion will involve an end-dumped closure section 50 feet high. Two basic problems are associated with closure embankments of this height - velocities during final closure would be quite high, requiring large size stone to remain in place, and subsequent sealing of the closure embankment in the wet must be done at significant depth, with relatively less control than for lower embankments.

In consideration of these problems and restraints, 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 this critical operation and avoiding potentially serious delays in the schedule. Two alternatives were re-evaluated as follows:

Tunnel Diameter (feet)	Upstream Cofferdam Crest Elevation (feet)	Upstream Cofferdam Approximate Height (feet)
30	1595	150
35	1545	100

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 feature. 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 (e), 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 1545 was incorporated as part of the selected general arrangement.

The various components of the selected diversion scheme are described in Section 12.

9.10 - Spillway Facilities Alternatives

As discussed in Section 9.7, the project has been designed to safely pass the floods with the following return frequencies:

Flood	Frequency	Spillway Discharge (cfs)
Design Flood	1:10,000 years	120,000
Probable Maximum Flood	--	235,000

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 flood conditions.

Clearly the selected spillway alternatives will greatly influence and be influenced by the project general arrangement. A discussion of the development of the general arrangement is presented in Section 9.12.

(a) Energy Dissipation

The two chute spillway alternatives considered effect energy dissipation either by means of a flip bucket which directs the spillway discharge in a free-fall jet into a plunge pool in the river well downstream from the structure, 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 excavated steps down to river level, with energy dissipation at each step and reduction of the velocity heads.

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 pressure relief drains and rock anchors in the foundation. A detailed description of the selected spillway alternative is given in Section 12.

(b) Environmental Mitigation

During development of the general arrangements for both Watana and Devil Canyon, a restriction was imposed on the allowance of excess dissolved nitrogen in the spillway discharges. Supersaturation occurs when aerated flows are subjected to pressure increases, forcing excess nitrogen into solution. This occurs when water is subjected to pressures approaching two atmospheres and would 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 extremely harmful effects on the fish population. On the basis of an evaluation of the related impacts, and discussions with interested federal and state environmental agencies, spillway facilities were designed to limit discharges of nitrogen supersaturated water from Watana to a recurrence period of not less than 1:50 years.

9.11 - 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.

The selection of the installed capacity of 1020 MW at Watana is described in detail in Section 9.6. The detailed comparison of power facilities alternatives is described in Appendix D. A summary of the general conclusions is described below.

(a) Comparison of Surface and Underground Powerhouse

Preliminary studies were carried out to compare the construction costs of a surface powerhouse and an underground powerhouse at Watana. These studies were undertaken on the basis of preliminary conceptual layouts assuming _____ units and an 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 predominantly 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 winder conditions in Alaska, is less affected by river flood floos in summer, and is aesthetically less obtrusive. This arrangement has therefore been adopted for further development.

(b) 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 generally required longer penstock and/or tailrace tunnels and were therefore strictly more expensive.

The location of the left bank was also not favored 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").

(c) Underground Openings

Cost estimates have been based on assumptions of full concrete lining of the penstocks and tailrace tunnels. The latter is a conservative assumption for preliminary design; in practice, a large proportion of the tailrace tunnels could 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 size of the larger excavation.

(d) Selection of Turbines

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

The number and rating of individual units is discussed in detail in Section 9.6. The final arrangement selected is six units of 170 MW rated at minimum reservoir level (from reservoir simulation studies) in the peak demand month (December) at full gate. The unit best efficiency output at rated head (680 feet) is 181 MW.

(e) Transformers

The selection of transformer type, size, location and step-up rating is described in Section 12.18 and 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, but, and transformer losses; and

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

(f) 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 (Sections 9.2 and 9.3).

The preferred penstock arrangement comprises six individual penstocks, one for each turbine. With this arrangement, no inlet valve is required in the powerhouse since penstock dewatering can be performed by using the control gate at the intake. An alternative arrangement with three penstocks was considered in detail to assess any possible advantages. This scheme would require 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 ($000) 6 Penstocks	3 Penstocks
Intake (increment)	-	-20.0
Penstocks (increments)	-	- 3.0
Bifurcations	-	+ 3.0
Valves	-	+ 4.0
Powerhouse	-	+ 8.0
Capitalized Value of Extra Head Loss	-	+ 6.0
Total	0	- 2.0

Despite a marginal saving of $2 million (or less than 2%) in favor of three penstocks, in a total estimated cost of $120 million, 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.

(g) Environmental Constraints

Apart from the potential nitrogen supersaturation problem discussed in Section 9.10, 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 through the anticipated range of drawdown in order to control the downstream river temperatures within acceptable limits.

Guaranteed 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 from the Susitna development. These studies are discussed in detail in Section 15.

In average to wet years, the Watana development will be capable of operating as a daily peaking plant for load following. The actual extent of daily peaking will be dictated by unit availability, system demand, unit generating costs, system stability, etc., (as described in Section 15). Predicted downstream water level fluctuation caused by daily peaking at Watana is within acceptable limits.

9.12 - 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

This comprised four steps:

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

- Step 2: Develop preliminary layouts based on the above data and design criteria 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 determined under Step 1c, taking into account the preliminary nature of the work at this stage.

(ii) Intermediate Review

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

- 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 conclusions from Step 1. 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 of 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 is presented in Table 9.5. Data and criteria developed during the preliminary and intermediate review stages are given in Appendix D for reference.

(c) Evaluation Criteria

The various layouts were evaluated at each stage of the review process on the basis of the criteria summarized in Table 9.6. The criteria listed in Table 9.6 illustrate the progressively more detailed evaluation process leading to the final selected arrangement.

9.13 - Preliminary Review

The development selection studies described in Section 8 involved comparisons of hydroelectric schemes at a number of sites on the Susitna River. For these comparisons a preliminary conceptual design was developed for the Watana project known as the "DSR Schemes".

Eight further layouts were subsequently prepared and examined for the Watana project during this preliminary review process, in addition to the DSR scheme. These eight layouts are shown in schematic form on Plate 13. Alternative 1 of these layouts was that recommended for further study in the Development Selection Report.

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

(a) 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 inter-connected 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 (see Section 9.2). Preliminary evaluation of the existing geological data indicated that the fracture materials and infill within the actual shear zones would be unable to support standard tunneling methods and would be inadequate for founding of massive concrete structures. The originally proposed dam axis was located between these shear zones, and as 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 as described in Section 9.8 was used as the basis for all layouts. The downstream slope of the dam was assumed as 2H:1V in all alternatives, upstream slopes varying between 2.5H:1V and 2.25H:1V were examined in order to determine the ifluence of variance in the dam slope on the congestion of the layout. In all these preliminary arrangements, except that prepared for the DSR, cofferdams were incorporated within the body of the main dam.

Floods greater than the routed 1:10,000 year spillway 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 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 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.

(b) Description of Alternative

(i) Preliminary DSR Scheme

The preliminary DSR scheme as shown on Plate 3 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 discharges 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, however, and the system would be very costly. The maximum head dissipated within each stilling basin is approximately 450 feet, within world experience, and 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 it could be located upstream almost as far as the system of drain holes and galleries just downstream of the main dam grout curtain.

(ii) Alternative 1

This alternative is that recommended for further study in the Development Selection Report and is similar to the preliminary DSR layout, 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 DSR 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 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.

(iii) 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 excavation but it is probable that most of this material, with carefule scheduling, could be used in the dam. The excavation would cross "The Fingerbuster" and extensive dental concrete would be required. 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. This 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 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 Alternative 1. However, the slope of the ground is less than the rather steep right bank and it 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 being more concentrated than the cascade 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, and 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.

(iv) <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 backup 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.

(v) 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.

(c) 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 of the preliminary DSR scheme 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:

(i) A variation of the preliminary DSR 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;

(ii) Alternative 2 with a right bank flip bucket spillway, an underground powerhouse on the left bank, and right bank diversion;

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

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

9.14 - 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 Plates 14 through 19. 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 design studies described in Section 12.

(a) Description of Alternative Schemes

The four schemes are shown on Plates 14 through 19:

(i) Scheme WP1 (Plates 14 and 15)

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 meansures 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 together with the underground powerhouse.

(ii) Scheme WP2 (Plates 16 and 17)

This scheme is derived from the DSR 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.

(iii) Scheme WP3

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 (see Plate 16).

(iv) Scheme WP4 (Plates 18 and 19)

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.

(b) 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 will 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 above the area of the relict channel, and this is closed by an approximately 50-foot high saddle dam. A slurry trench cutoff will be combined with grouting to seal the 200-foot depth of pervious material infilling this channel.

A summary of capital cost estimates for the four alternative schemes is given in Table 9.7.

The results of this intermediate analysis indicate that the chute spillway with flip bucket of 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 tha 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 absolutely 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 9.7.

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.

(c) 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 9.7, 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 route 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, as discussed in Section 9.10. A cascade configuration would reduce the dissolved nitrogen content, and 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 fruther evaluation as part of the final review process. These schemes are described separately in the paragraphs below.

9.15 - Final Review

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

(a) Scheme WP 3 A (Plate 20)

This scheme is a modified version of Scheme WP 1 described in Section 9.14 with an emergency spillway as included in Scheme WP 3. 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 "Fingerbuster." The upstream cofferdam and main dam are maintained in the upstream locations as shown on Plate 14. 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:

(i) Main Dam

Further investigation of preliminary design studies and review of world practice suggests 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.

(ii) 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 directions of river flow and diversion inflow and outflow at the portals.

A separate low-level inlet and concrete-lined tunnel is provided leading from the reservoir at approximate elevation 1550 feet to downstream from the diversion plug where it merges with the diversion tunnel closest to the river. This low-level tunnel is designed to pass flows in excess of 2000 cfs acting as a low-level release during reservoir filling. It will also pass up to 10,000 cfs under 500-foot head to allow emergency draining of the reservoir as discussed in Section 9.9.

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 gates within the second portal are lowered and a mass concrete closure plug constructed on a line with the grout curtain. After closure of the gates, filling of the reservoir can commence.

(iii) Emergency and Outlet Release 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 deptch adjacent to the power facilities intake structures. Flows will then be passed downstream through a hired shaft and tunnel, existing beneath the downstream end of the main spillway flip bucket, as described below. In order to overcome potential nitrogen supersaturation problems, Scheme WP 3 A also incorporates a system of fixed core valves into the downstream end of the emergency release facility. The valves were sized to discharge in conjunction with the powerhouse operating at 75 percent capacity, flows up to the equivalent routed 50-year flood. Six 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.

(iv) Spillways

As discussed in Section 9.10 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 of not less 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 release and power intake structures just upstream from the dam centerline. The design discharge is approximately 80,000 cfs corresponding to the routed 1:10,000-year flood (120,000 cfs) reduced by the 40,000 cfs flows attributable to outlet release and power facilities discharges. The plunge pool is formed by excavating the alluvial river deposits to bedrock; and as this approaches the limits of the calculated maximum scour hole, it is not anticipated that, given the infrequent discharges, significant downstream erosion will occur.

The 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 feet, 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 235,000 cfs.

(v) Power Facilities

The power intake is set slightly upstream from the dam centerline 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 is affected 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 nearby 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 contain 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, area 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 upstream. 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 the main alignment of the excavations running parallel to the major joint sets as described in Section 9.3.

(vi) 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.

(b) Scheme WP 4A (Plate 21)

This scheme is similar in most respects to Scheme WP 3A previously discussed, except for the spillway arrangements.

(i) Main Dam

The main dam axis is similar to that of Scheme WP 3A, except for a slight downstream rotation at the left abutment at the spillway control structures.

(ii) Diversion

The diversion and low head releases are exactly similar for the two schemes.

(iii) Emergency and Outlet Release Facilities

The emergency drawdown release facility is separated from the main spillway for this scheme. The emerging relase consists of a low-level gated outlet structure located upstream, discharging up to 30,000 cfs well 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.

(iv) 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 Plate 21 for Scheme WP 4A 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.

(v) Power Facilities

The power facilities are similar to those in Scheme WP3A.

(c) 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 WP 3A. If this difference is adjusted for the savings associated with using an appropriate proportion of excavated material 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 of the left bank, it is known that a major shear zone runs through and is adjacent to the area presently allocated to the spillway. 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.

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. As this scheme presents a more economic solution with fewer potential problems concerning the geotechnical aspects of its design, the right bank chute arrangement has been adopted as the final selected scheme.

TABLE 9.3: 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)
2,240	3,542
2,190	3,322
2,140	3,071

* Estimated costs in January 1°82 dollars, based on preliminary conceptual designs, including relict channel drainage blanket and 20 percent contingencies.

TABLE 9.4: LONG-TERM PRESENT WORTH

Watana Dam Crest Elevation (ft MSL)	Long Term System-Present* Worth ($ x 10^6)
2240 (reservoir elevation 2215)	7,123
2190 (reservoir elevation 2165)	7,052
2140 (reservoir elevation 2115)	7,084

* January 1982 dollars.

TABLE 9.5: 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):	235,000 cfs
Maximum inflow with return period of 1:10,000 years:	155,000 cfs
Maximum 1:10,000-year routed discharge:	120,000 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:	2,215 ft
Reservoir minimum operating level:	2,030 ft

Dam

Type:	Rockfill
Crest elevation at point of maximum super elevation:	2,240 ft
Height:	890 ft above foundation
Cutoff and foundation treatment:	Core founded on rock; grout curtain and downstream drains
Upstream slope:	1V:2.4H:1V
Downstream slope:	1V:2H:1V
Crest width:	50 ft

Diversion

Cofferdam type:	Rockfill
Cutoff and foundation:	Slurry trench to bedrock
Upstream cofferdam crest elevation:	1,585 ft
Downstream cofferdam crest elevation:	1,475 ft
Maximum pool level during construction:	1,580 ft
Tunnels	Concrete lined,
Final closure:	Mass concrete plugs
Releases during impounding:	6,000 cfs minimum 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 9.5: (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

Powerplant

Type of turbines:	Francis
Number of 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:	148 MVA
Power factor:	0.9
Frequency:	60 HZ
Transformers:	148 MVA - 13.8-345 kV, 3-phase

Tailrace

Water passages:	2 concrete-lined tunnels
Surge:	Separate surge chambers
Average tailwater elevation (full generation):	1,458 ft

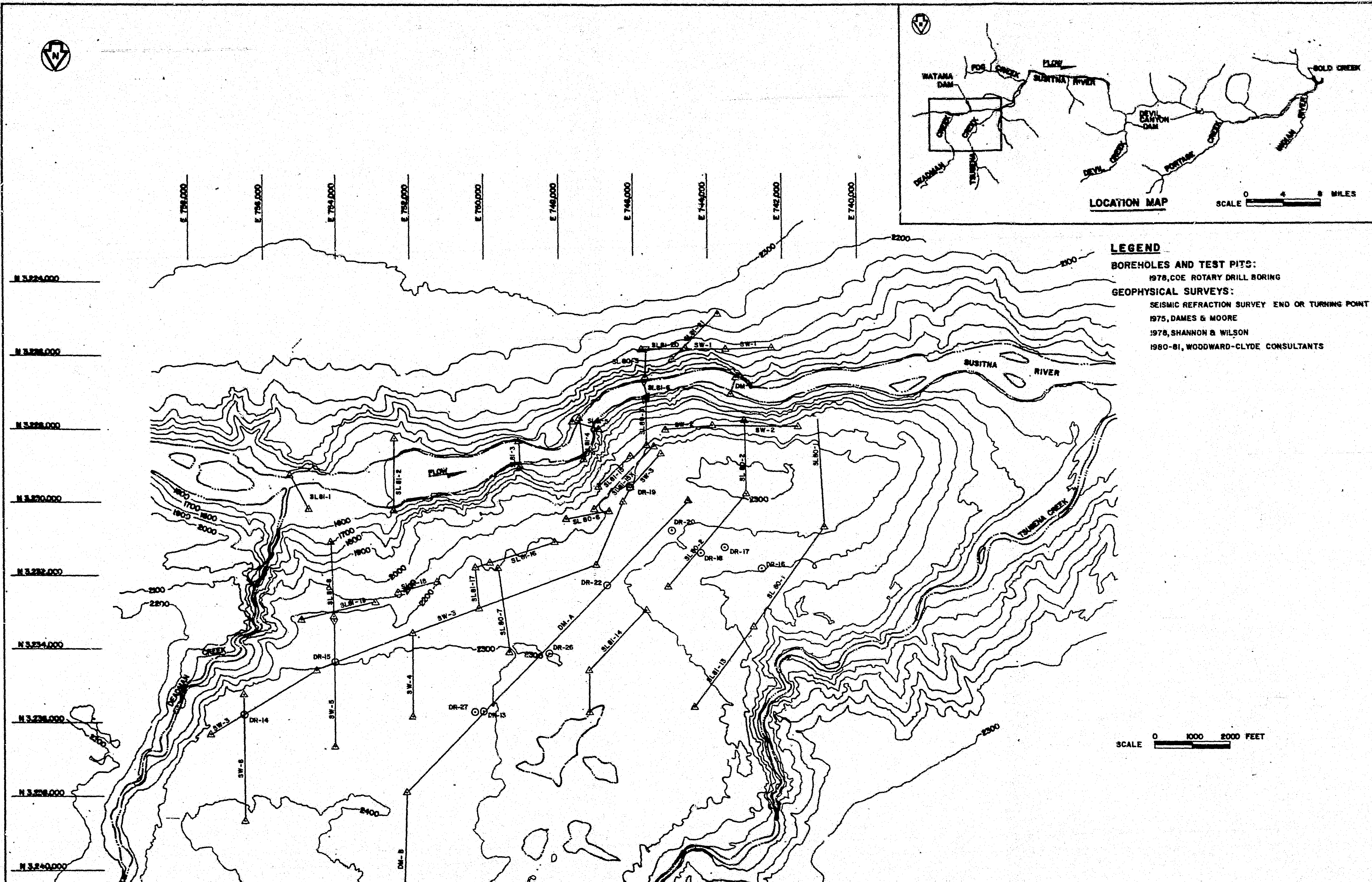
TABLE 9.6: EVALUATION CRITIERA

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 9.7: SUMMARY OF COMPARATIVE COST ESTIMATES

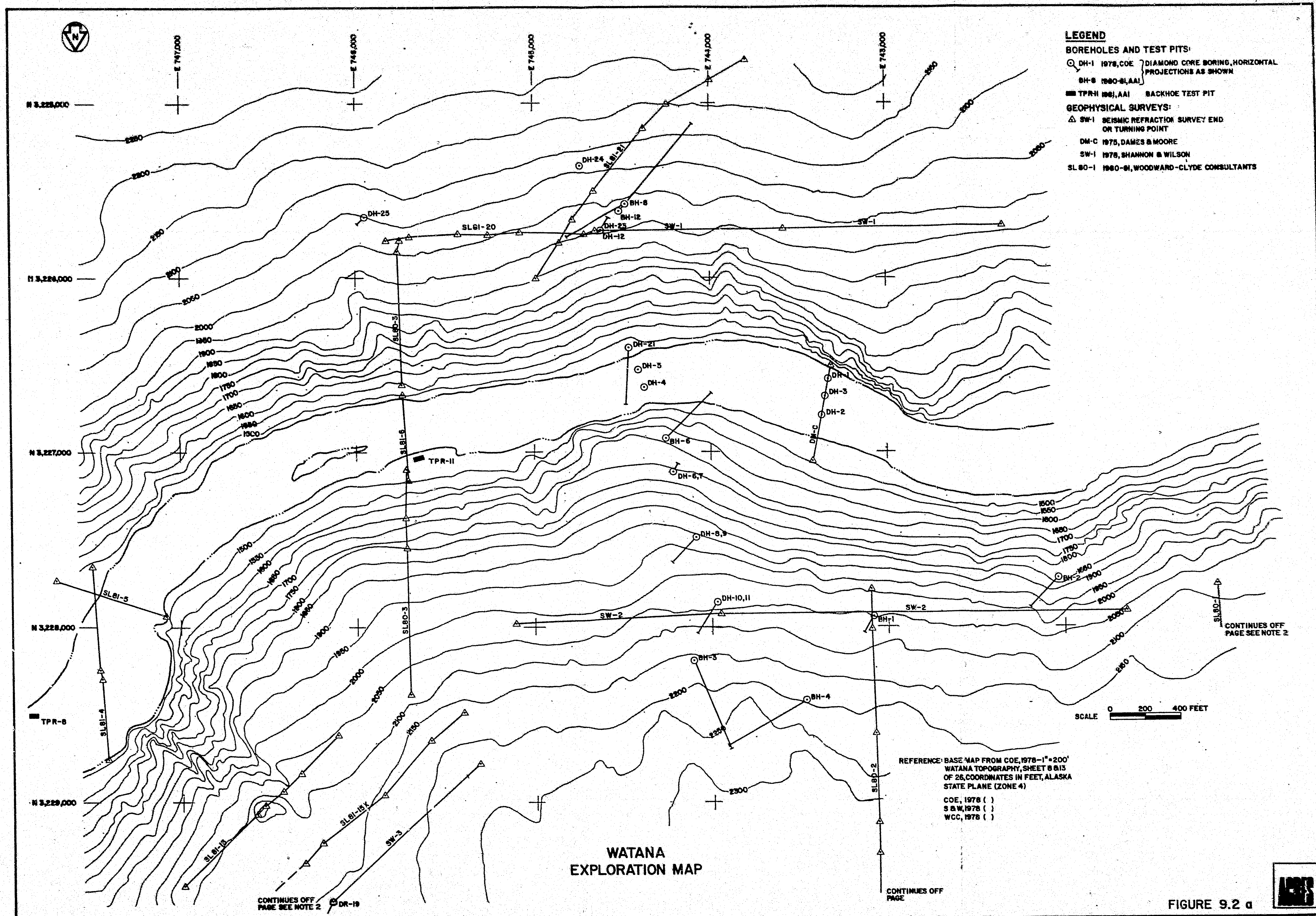
INTERMEDIATE REVIEW OF ALTERNATIVE ARRANGEMENTS
(January 1982 $000,000)

	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



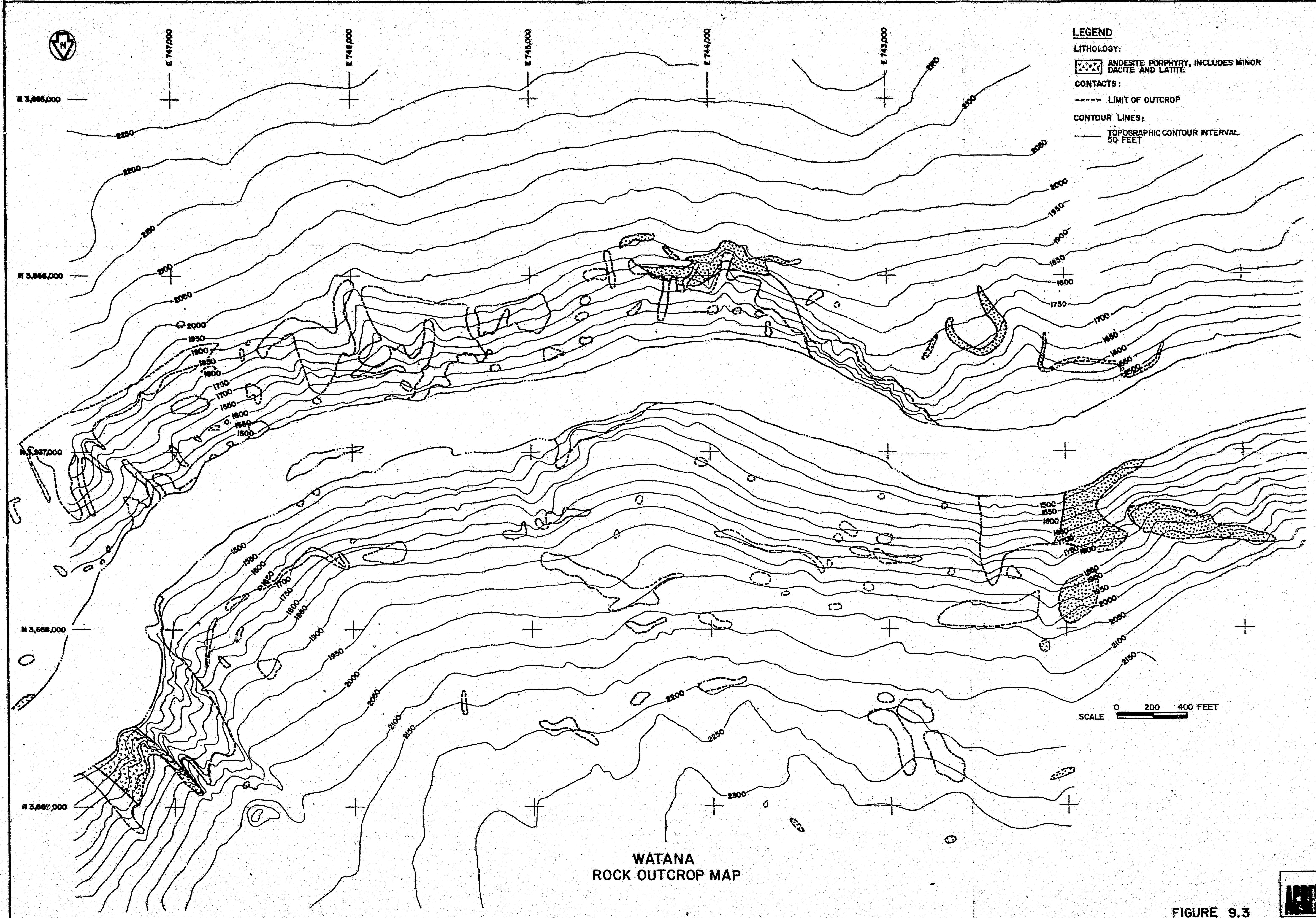
WATANA
DAMSITE VICINITY-EXPLORATION MAP

FIGURE 9.2



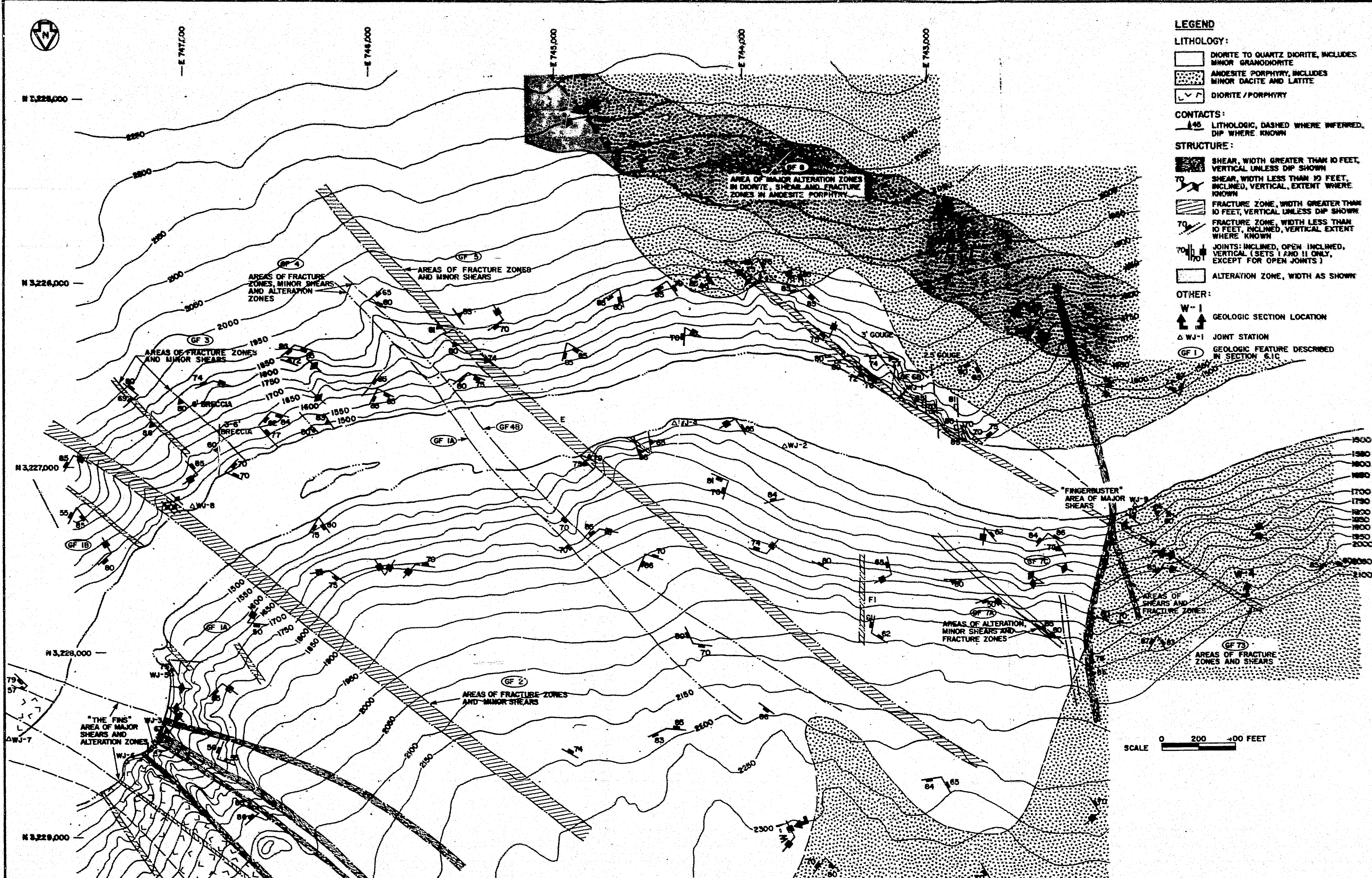
WATANA
EXPLORATION MAP

FIGURE 9.2 a



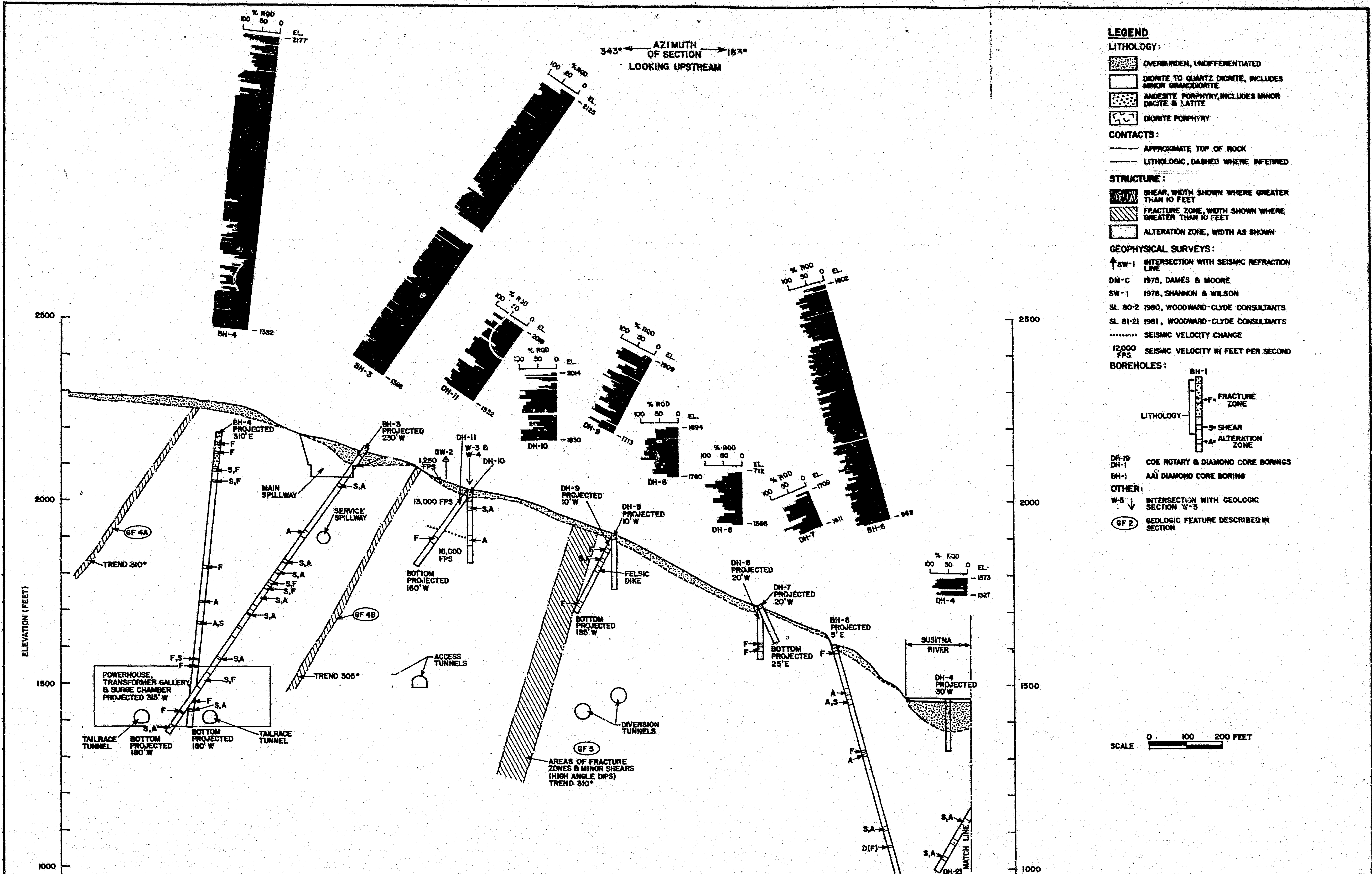
WATANA
ROCK OUTCROP MAP

FIGURE 9.3



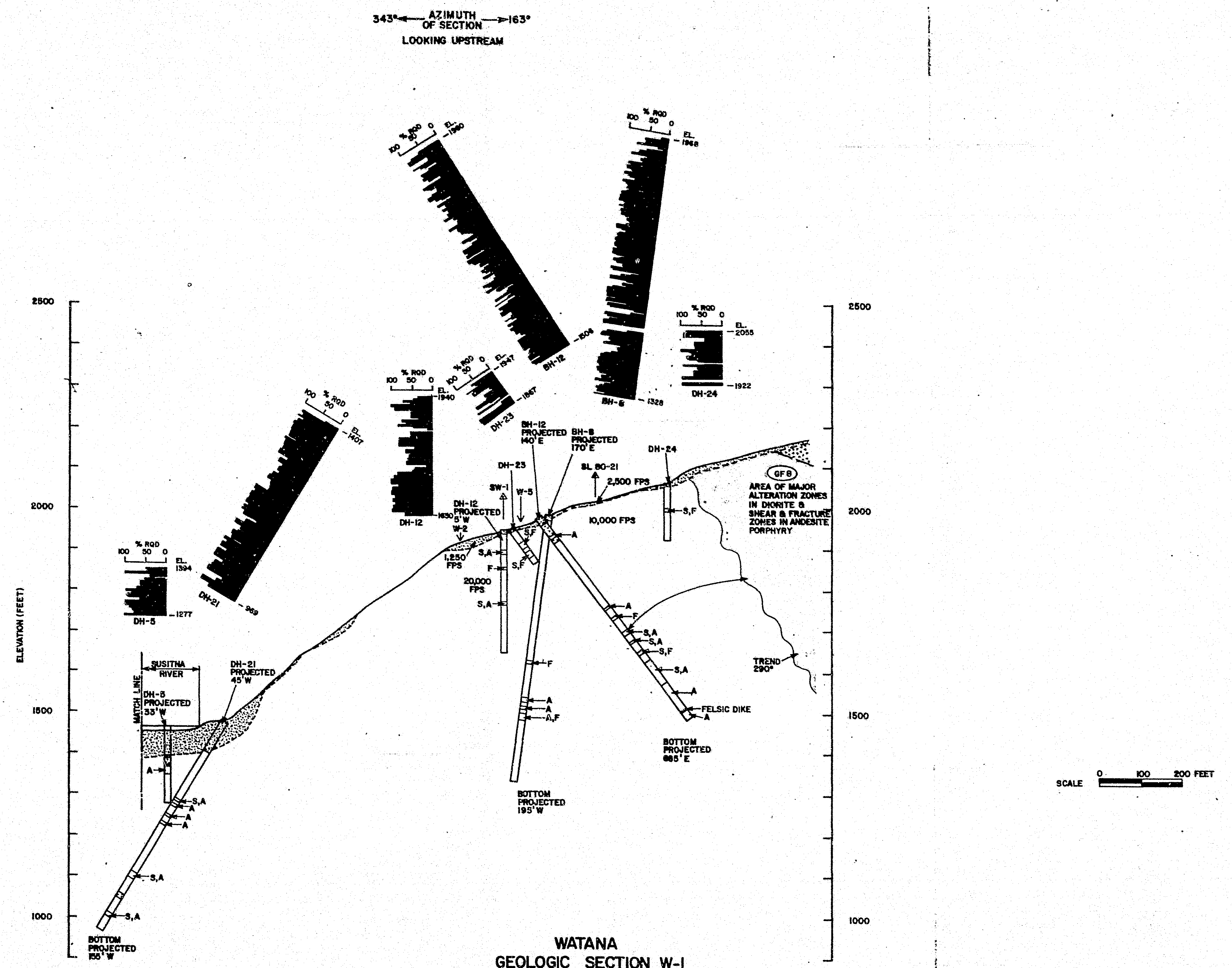
WATANA
GEOLOGIC MAP

FIGURE 9.4



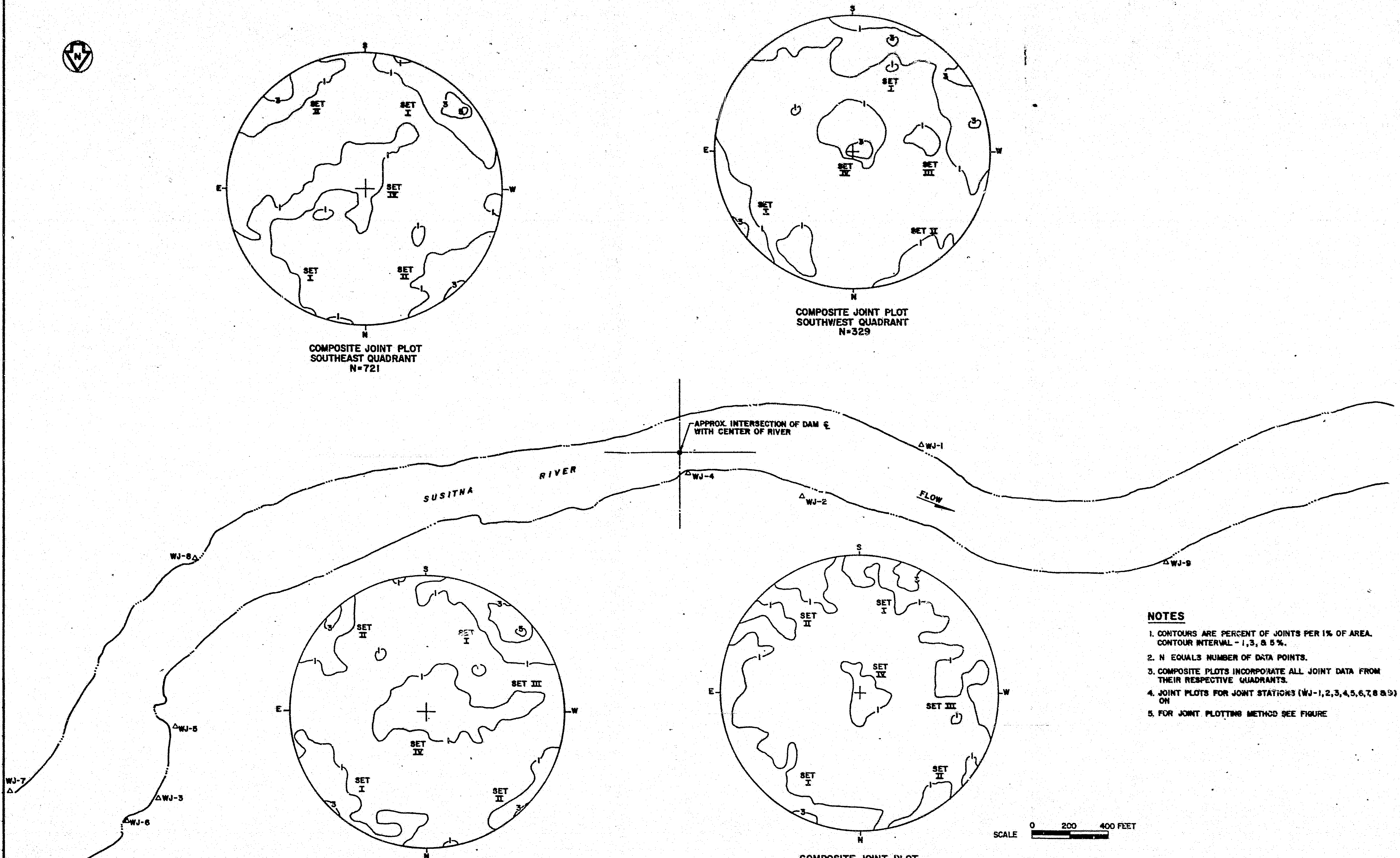
WATANA
GEOLOGIC SECTION W-1
SHEET 1 OF 2

FIGURE 9.11



WATANA
GEOLOGIC SECTION W-1
SHEET 2 OF 2

FIGURE 9.11a

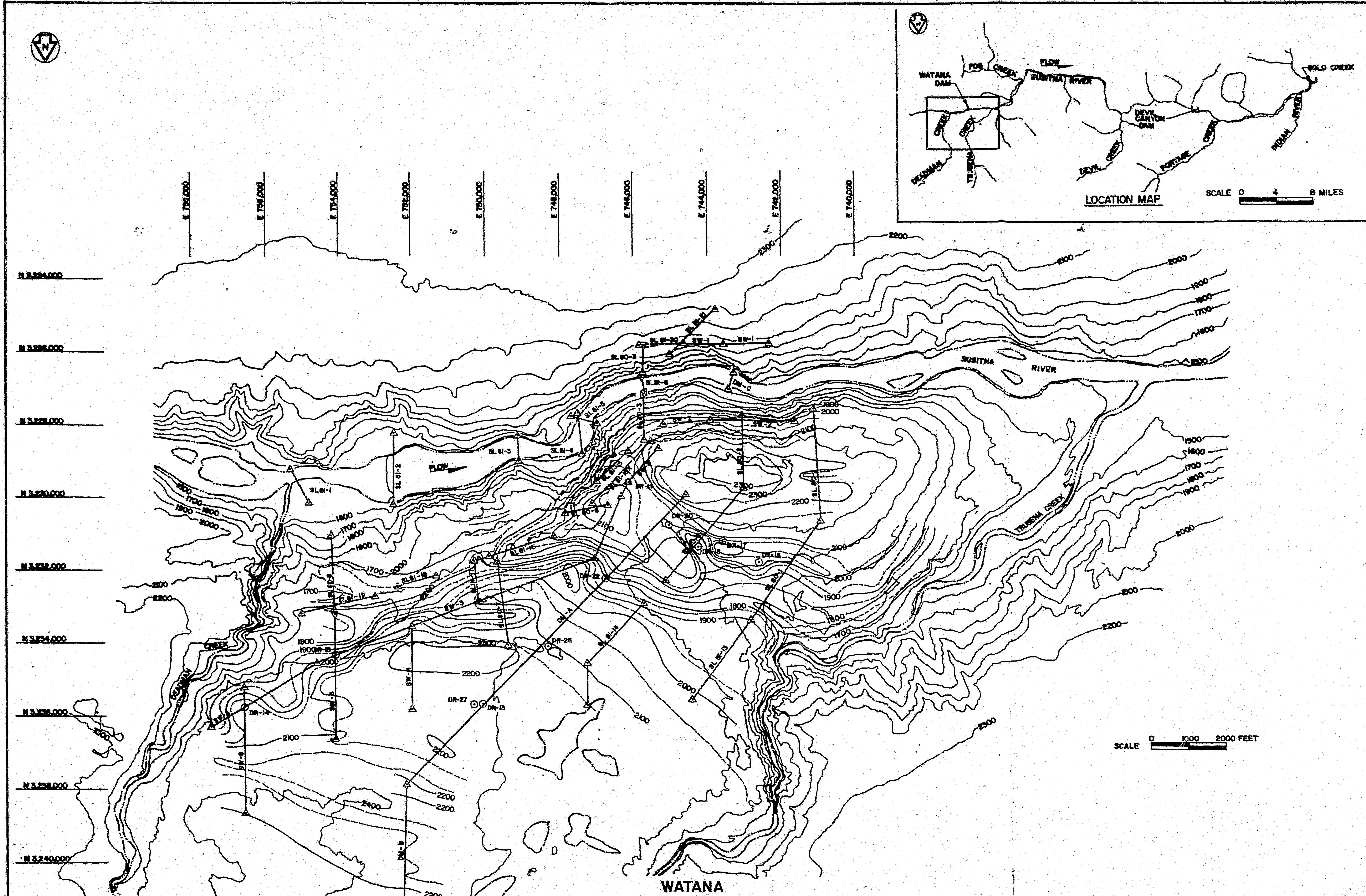


NOTES

1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1, 3, & 5%.
2. N EQUALS NUMBER OF DATA POINTS.
3. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA FROM THEIR RESPECTIVE QUADRANTS.
4. JOINT PLOTS FOR JOINT STATIONS (WJ-1,2,3,4,5,6,7,8 & 9) ON
5. FOR JOINT PLOTTING METHOD SEE FIGURE

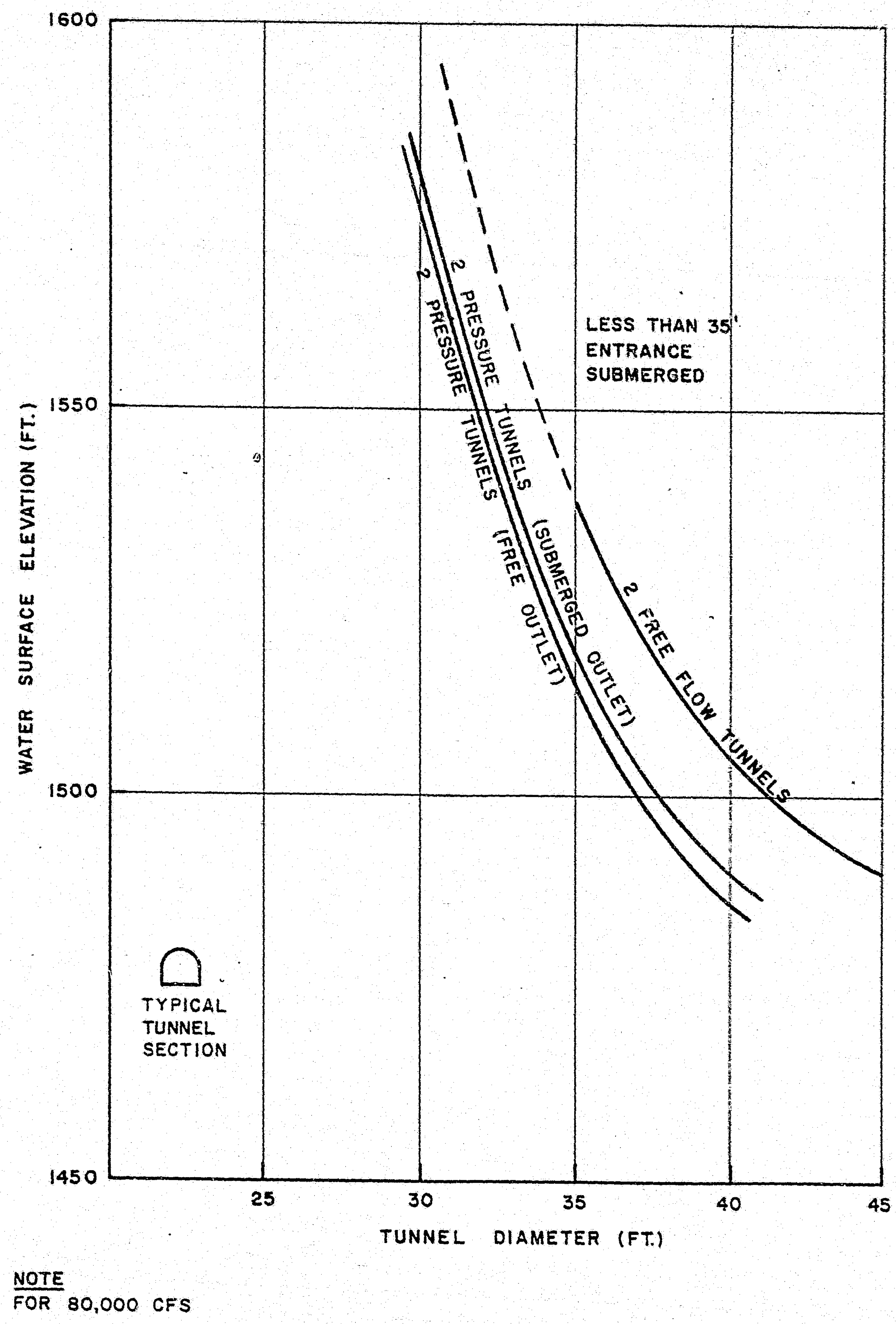
WATANA
COMPOSITE JOINT PLOTS

FIGURE 9.12



WATANA
RELICT CHANNEL TOP OF BEDROCK

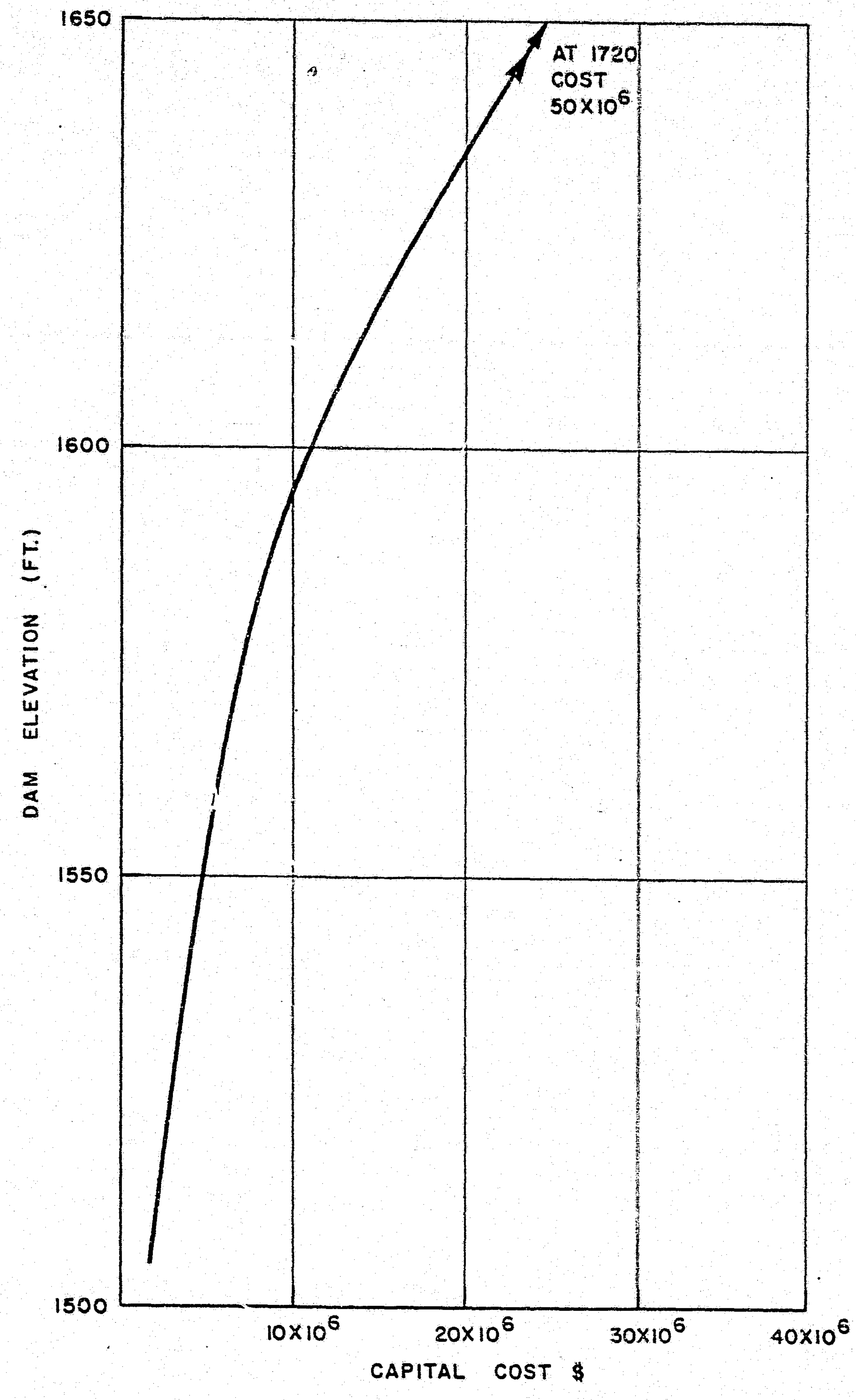
FIGURE 9.12 a



WATANA DIVERSION
HEADWATER ELEVATION / TUNNEL DIAMETER

FIGURE 9.13

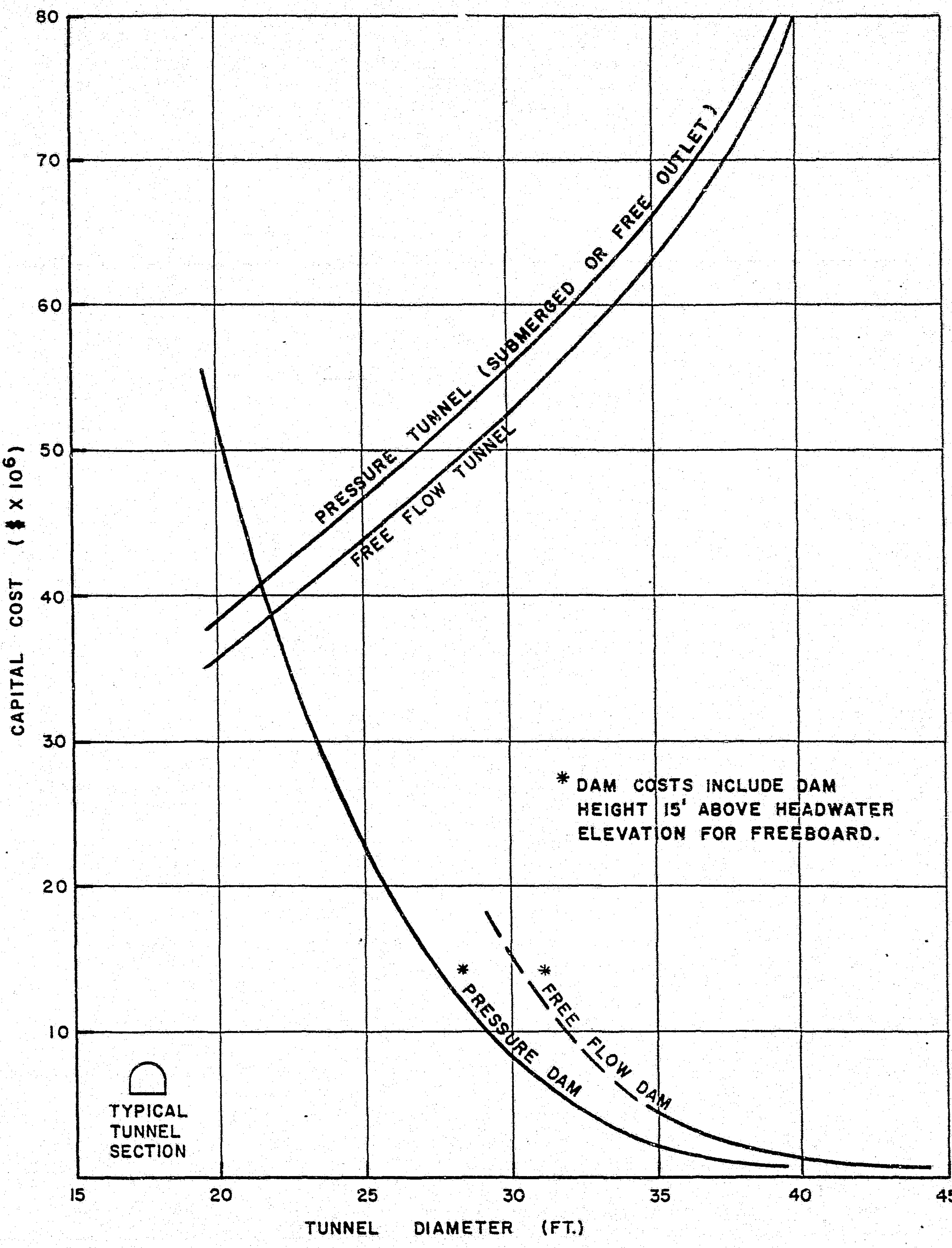
ACRES



WATANA DIVERSION
UPSTREAM COFFERDAM COSTS

FIGURE 9.14

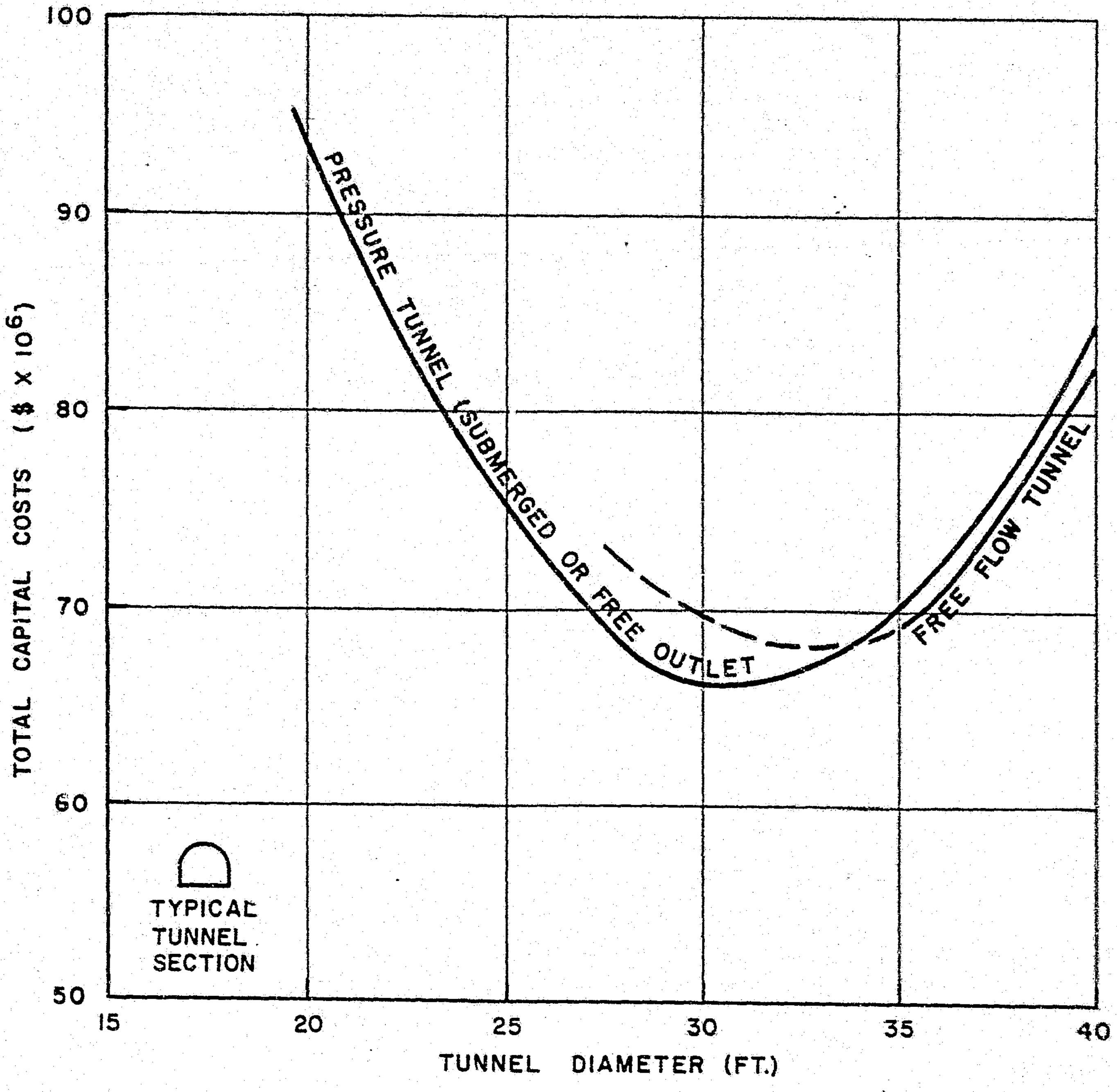
ACRES



WATANA DIVERSION

TUNNEL & COFFERDAM COST / TUNNEL DIAMETER

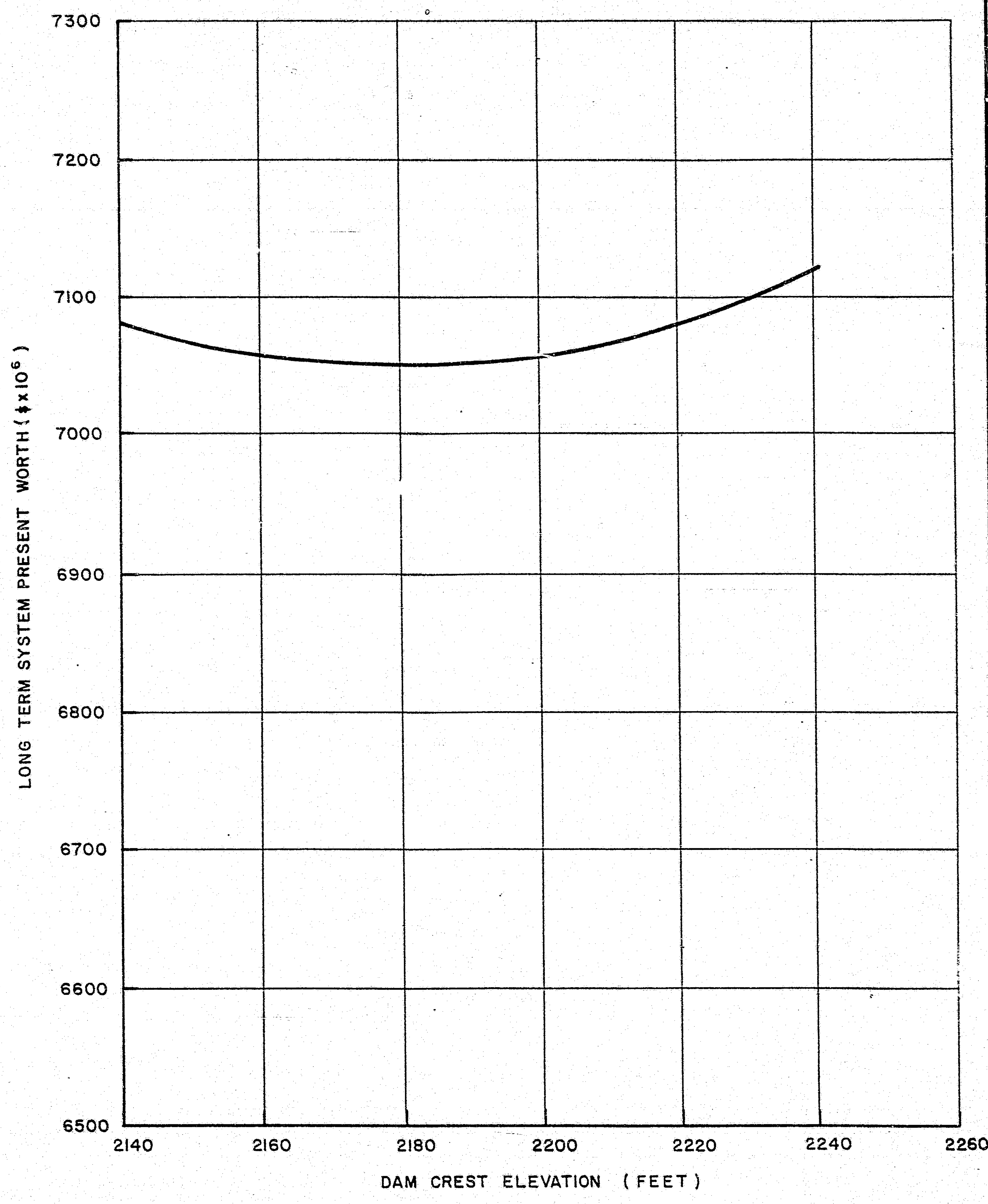
FIGURE 9.15



WATANA DIVERSION
TOTAL COST / TUNNEL DIAMETER

FIGURE 9.16

ACRES



SELECTION OF RESERVOIR LEVEL

ACRES

FIGURE 9.17

10 - SELECTION OF DEVIL CANYON GENERAL ARRANGEMENT

This section describes the development of the general arrangement of the Devil Canyon project. The site topography, geology, and seismicity of the Devil Canyon site are described relative to the design and arrangement of the various site facilities, in a manner similar to that presented in Section 9 for the Watana site. 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 detailed description of the various project components is given in Section 13.

10.1 - Site Topography

The Devil Canyon site is located at river mile 152 of the Susitna River, approximately 31 miles downstream from the Watana site, in a "V" shaped section near the entrance to the canyon which is about 2 miles long. The valley wall on the left side of the river rises very steeply from Elevations 900 to 1300 on the left bank at a slope of approximately 0.4H:1V to a relatively gently sloping plateau area which reaches Elevation 1600 within the general project area. On the right side, the valley is less pronounced, rising at about 1.1H:1V to Elevation 1500, then much more gradually to approximate Elevation 1900. The steep left bank features overhanging cliffs and detached blocks of rock.

10.2 - Site Geology

This section summarizes the geological and geotechnical investigations and interpretations conducted to date and the conditions present at the proposed Devil Canyon site. The detailed description of the site investigations and the geologic and geotechnical conclusions are presented in the 1980-81 Geotechnical Report (1).

(a) Geologic Setting

Devil Canyon has been eroded through hard metamorphosed sedimentary rocks, argillite and graywacke of excellent quality (Figure 7.11). The bedding strikes roughly parallel to the river and dips to the south. Overburden is generally thin to nonexistent. Stress relief cracks and open joints parallel the gorge and extend more than 100 feet from the canyon walls.

On the left (south) bank, a series of small lakes parallel the valley. Deep overburden up to 80 feet thick has been encountered in this area which probably represents an old stream buried under glacial material. A highly sheared and fractured zone is present under this buried stream. Work performed during this study, however, showed this feature to be of no seismic concern (3).

A large alluvial fan exists at the confluence of Cheechako Creek with the Susitna River, about 1,000 feet upstream from the damsite. This area is the main source of material for the concrete aggregates and the filter materials for the saddle dam.

(b) Geological and Geotechnical Investigations

Surface and subsurface investigations have been conducted by several organizations at different times. During the period from June 1957 to August 1958, the USBR conducted geologic mapping and subsurface investigations at this site. The subsurface investigations included drilling of rock and overburden, in-hole testing, test pits, and laboratory tests. Subsequently, in 1978, the COE conducted seismic refraction surveys to expand on this work to assess the suitability of the site. During the years 1980 and 1981, more detailed investigations of geologic features were performed as part of the current work program to establish the technical feasibility of the project. These investigations have included air reconnaissance, air photo interpretation, geologic mapping of rock and overburden including in-hole geophysical tests, and seismic refraction surveys. Both in situ and laboratory tests have been performed to determine the engineering characteristics of soils and rocks. The location of drill holes, test pits, and other investigations is shown in Figure 10.1.

Geologic mapping was concentrated in the immediate damsite area to define the geology of the site in as much detail as possible. Under extremely difficult conditions of accessibility, ground mapping traverses were accomplished by making maximum use of technical climbers in the gorge to augment mapping activities on foot along the upper slopes. At each station, the applicable lithology or type of overburden, bedding, jointing, weathering, degree of consolidation, exposure size, and elevations was noted and plotted on maps for use in the interpretation. All accessible areas with rock outcrops were mapped.

Seismic refraction surveys totalling 3,300 feet were performed on the south side of the canyon across the lake area and the alluvial fan to delineate their extent and characteristics.

Diamond core drilling was performed on the upper slopes and at river level of the canyon. The holes were cased through the overburden leaving access for future testing and instrumentation. A total of 4,800 linear feet in 29 holes have been drilled at the Devil Canyon site. Comprehensive logs have been developed for each hole. Water pressure testing using inflatable packers was conducted in the holes for permeability calculations. Geophysical logging and borehole photography were attempted during the 1980 season in selected holes.

To monitor the ground water and ground temperature conditions at the site, piezometers and thermistors were installed in selected drill holes. Periodic readings of these instruments, after stabilization, have been conducted to give more detailed information of the conditions that may be encountered during construction.

A series of tests was performed on the rock recovered from coring to determine the engineering characteristics of the rock mass. The results of these investigations were compiled, correlated, and interpreted to develop the geologic picture of the damsite and the adjacent areas. The results of the laboratory rock tests are summarized in Table 10.1.

(c) Construction Material Investigations

Most of the investigations for the construction materials at the Devil Canyon site were performed during the years 1957 and 1958. Additional exploration during the years 1980 and 1981 was undertaken to supplement the previous work. The investigations have included geologic mapping, auger drilling, excavation of test pits and test trenches, seismic refraction surveys, and laboratory tests. A total of 8 auger holes, 2 test pits, and 4,100 linear feet of seismic refraction survey have been completed. The location of the various sources for the concrete aggregate and the saddle embankment dam are shown in Figure 10.1.

A major source of construction materials for the Devil Canyon Project is an alluvial fan of deposits, which lies near the Cheechako Creek confluence approximately 1,000 feet upstream from the arch damsite. The area contains large quantities of sands and gravels with inclusions of boulders and cobbles above the river level. Under a thin mantle of organic material, a 3-to-4-foot layer of silty sand overlies a layer of sandy gravel with traces of silt and some cobbles and boulders. This layer of sandy gravel is about 80 feet thick. With proper processing, this source will provide coarse and fine aggregate for the concrete, filters for the embankment dam and cobbles for the upstream shell of the embankment dam. A composite grain-size curve for this material is presented in Figure 10.2.

(i) Rockfill Material for the Saddle Dam

The required quantities of rockfill can be obtained from the area designated Quarry Site K. The rock in this site is primarily diorite. It is hard, durable and fresh, and suitable for the embankment construction. Also, the suitable portions of the rock excavated for the foundation of the arch dam and other project structures, such as underground power facilities and main and emergency spillways, are adequate for use in the embankment, subject to appropriate scheduling of excavation and fill operations. Sufficient fill quantities are available at the site to meet all requirements.

(ii) Impervious Core Material for the Saddle Dam

No suitable source for the core material for the saddle dam has been identified at this time near the site. For current feasibility assessment purposes, it is planned that the core material will be transported from Borrow Area D near the Watana site, where sufficient quantities of suitable material have been identified. A discussion of the engineering characteristics of those materials is presented in Section 9. Additional investigations will be performed in the future in an attempt to locate a potential source nearer the Devil Canyon site.

(iii) Filter Material for the Saddle Dam

Filter and transition zone materials will be obtained from the alluvial fan Borrow Area, as discussed above.

(iv) Gravels and Cobbles for the Saddle Dam

If needed, sufficient quantities of clean gravel and cobbles can also be obtained from the alluvial fan with proper processing.

(v) Concrete Aggregate

The coarse and fine aggregate for the concrete structures will also be obtained from the alluvial fan upstream. The results of the laboratory testing, presented in Table 10.1, indicate that the material from this source is of adequate quality. The gravel particles are generally rounded with accompanying subangular sands. Petrographic analyses indicate that the material includes quartz diorites, granites, andesites, diorites, dacites, metavolcanics, rocks, aplites, breccias, schists, phillites, arqillites, and amphibolites. Generally, the material has less than 2 percent deleterious constituents such as chert, muscovite, and argillite.

(d) Geologic Conditions

The overburden and bedrock conditions at the Devil Canyon site are discussed in this section.

(i) Overburden

The valley walls at the Devil Canyon site are very steep and are generally covered by a thin veneer of overburden consisting primarily of talus at the base (Figure 10.3). The flatter upland areas are covered by 5 to 35 feet of overburden of glacial origin. The topographic depression along the elongated lakes on the south bank has an overburden covering in excess of 85 feet of glacial materials. The overburden on the alluvial terrace or point bar deposit at the Cheechako Creek confluence thickens from 100 feet to more than 300 feet over a distance of less than 400 feet.

The river channel alluvium appears to be composed of cobbles, boulders, and detached blocks of rock and is inferred to be up to 40 feet thick. A representative cross section across the valley is presented in Figure 10.4.

(ii) Bedrock Lithology

The bedrock at the Devil Canyon site is a low-grade metamorphosed sedimentary rock consisting predominantly of argillite with interbeds of graywacke (Figure 10.5). The argillite is a fresh, medium-to-dark gray, very thinly bedded, very fine grained argillaceous rock with moderately well-developed foliation parallel to the bedding. The graywacke is a fresh, light gray, mainly fine grained sandstone with an argillaceous matrix. It is locally a conglomerate with lithic fragments up to two inches in size. The graywacke is well indurated and exhibits poorly developed to non-existent foliation. The graywacke is interbedded with the argillite in beds generally less than 6 inches thick. Contacts between beds are tight and both rock types are fresh and hard. Minor quartz veins and stringers have intruded the argillite. These are generally less than 1 foot wide and unfractured with tight contacts. Sulphide mineralization is common with pyrite occurring in as much as 5 percent of the rock.

The area has also been intruded by numerous felsic and mafic dikes ranging from 1 inch to 60 feet wide (averaging 20 feet). The dikes have northwest to north orientation (Figure 10.5) with steep dips. When closely fractured they are easily eroded and tend to form steep talus-filled gullies, some of which exhibit shearing with the host rock. The felsic dikes are light gray silicic varieties including aplite and rhyolite. The mafic dikes are fine grained and appear to be of diorite to diabase composition.

(iii) Bedrock Structures

- Bedding

The argillite/graywacke has been completely deformed as evidenced by refolded folds and the development of multiple foliations. The primary foliation parallels the bedding at 35° to 90° (N35E to E), subparallel to the river, and dips 45° to 80°SE. When two or more foliations are parallel, the rock has a very slatey/phyllitic appearance, and when oblique, the rock appears massive. The canyon at the damsite appears to be controlled by the bedding planes.

- Joints

Four joint sets have been delineated at Devil Canyon, as shown on the stereo plots (Figure 10.6). Set I (strike 320° to 355° and dip 60NE to 70NW) and Set II (strike 040° to 065° and 40° to 60°S dips) are the most significant. Set I joints are the most prominent with spacing of 15 feet to 2 feet, and on the upper canyon walls of the south bank are open as much as 6 inches (Figure 10.5). Set III is subparallel to the bedding/foliation and, when it intersects with Set I, can cause the formation of loose blocks. Set III joints (strikes 005° to 030° and dips 85NW to 85SE) are also often open on the south bank and may dip towards the river, creating potential slip planes. This set, however, has variable spacing and sporadic distribution. The fourth set is a minor set with low dip angles and variable strike orientation.

Joint spacings measured from the borehole cores range from less than 1 foot to more than 10 feet. The spacing and tightness of the joints increase with depth, and the iron oxide staining and weathering extends up to 80 feet.

- Shears and Fracture Zones

Shears and fracture zones were encountered in localized areas of the site in both outcrops and boreholes. Correlation of the data is shown on the interpretation map (Figure 10.5). Shears are defined as areas containing breccia, gouge, and/or slickenslides indicating relative movement. These zones are soft and friable and are characterized by high permeability and core loss during drilling. Fracture zones, often encountered in conjunction with the shears, are zones of very closely spaced joints. With depth, these zones become smaller, tighter, and more widely spaced. Where exposed, they are eroded into deep gullies.

The most common trend of these features is northwest, parallel to Joint Set I. These have vertical to steep northeast dips and are generally less than 1 foot wide. Northwest trending shears are also associated with the contacts between the argillite and mafic dikes and are up to 1 foot wide, with closely spaced joints prominent in the dike itself.

A second series of shears trend northeasterly, subparalleling the bedding/foliation and Joint Set II, and have high angle southeasterly dips. These average less than 6 inches in width.

(e) Structural Features

Several structural features at the Devil Canyon site were investigated during the 1980-81 program.

In summary, these included the east-west trending sheared and fractured zone beneath the proposed saddle dam area; a bedrock drop-off beneath Borrow Site G; and bedrock conditions beneath the Susitna.

Seismic refraction and drilling data confirm the existence of a highly sheared and fractured zone on the left bank beneath the proposed saddle dam that generally trends parallel to the river. The dip on this feature is inferred to be parallel or subparallel to the bedding/foliation at approximately 65° to the south. The linear extent of the feature has been inferred to be approximately 2,500 feet. No evidence was found during the 1980-81 program to suggest movement along this feature. This finding was also concluded by work done by Woodward-Clyde Consultants (3). Further investigation of this feature will be required to define its extent and type of foundation treatment that will be required beneath the saddle dam in subsequent phases of investigation.

Upstream from the damsite, a several-hundred-foot drop-off in bedrock surface was detected by seismic refraction surveys under the alluvial fan. Land access restrictions imposed during the study prohibited any further investigation of this area. Possible explanation for this apparent anamalous drop-off could be attributed to misinterpretation of the seismic data or else the lower velocity material could be either a highly fractured rock in lieu of soil or an offset of the rock surface caused by faulting. The latter interpretation is unlikely in that work performed by Woodward-Clyde Consultants (3) in this area concluded that there was no compelling evidence for a fault. Future work remains to be done in this area to more clearly define this feature.

Detailed examination of rock core and mapping in the river valley bottom showed no evidence for faulting in the riverbed.

(f) Ground Water Conditions

Ground water migration within the rock is restricted to joints and fractures. It is inferred that the ground water level is a subdued replica of the surface topography with the flow towards the river and lakes. Measured water levels in the boreholes range average approximately 120 feet below surface.

(g) Permafrost

Preliminary temperature measurements made in the boreholes did not encounter permafrost conditions on either side of the river.

(h) Devil Canyon Reservoir Geology

The Devil Canyon reservoir will be confined to a narrow canyon where the topography is controlled by bedrock. The overburden is thin to nonexistent, except in the upper reaches of the reservoir where alluvial deposits cover the valley floor. Near the Watana site, light gray to pink, medium grain diorite rock is present. This rock is hard, massive, and competent except on the upland north of the Susitna River where the biotite granodionite has been badly weathered. The principal rock types in the most part of the reservoir are the argillite and graywacke which are exposed at the damsite. The rock has been isoclinically folded into steeply dipping structures striking generally northeast-southwest. The argillite has been intruded by massive granodiorite, and as a result, large isolated roof pendants of the argillite and graywacke are found locally throughout the entire reservoir and surrounding areas. The joint measurements at selected areas indicate structural trends similar to those at the damsites.

10.3 - Geotechnical Considerations

The geotechnical investigations to date have been primarily directed toward the important geological features which may have significant impact on the feasibility of the project. More detailed investigations, including exploratory adits, will be required prior to the detailed design.

(a) Arch Dam Foundation and Abutments

The geologic and topographic conditions are favorable for an arch dam at the Devil Canyon site. The rock is principally hard, competent, and fresh with weathering limited to joints and shear zones. Intrusive mafic and felsic dikes, where present, are hard, and the contact with the parent rock is tight. The orientation of these dikes is generally NW to N and has no important adverse effect on the stability of the abutments. The unconfined compressive strength of the intact rock ranges between 16,000 psi and 32,000 psi. The stresses imposed by the arch dam are about 1,000 psi or less under normal conditions. Even under extreme loading conditions, the stresses will be well within the acceptable limits for bearing considerations. On the right abutment, the arch dam thrust block will be seated in good sound rock. The top of the hill is at approximately Elevation 1500 and no major rock discontinuity is present. However, on the left side, massive thrust blocks will be required to transfer the loads to competent rock and to form an abutment to the saddle dam.

Essentially no continuous, poorly oriented rock discontinuities, which might adversely affect abutment stability, have been found. The major joint set at the site, influencing the stability of the south bank abutment, strikes approximately northwest with a near vertical average dip. The stability of the right abutment (north bank) is controlled by the bedding planes and foliations that strike roughly parallel to subparallel to the canyon walls and dip steeply into the canyon. The bedding planes generally appear to be tight with undulating surfaces because of the extensive folding of the rock. Preliminary analyses indicate no stability problems. Additional rock investigations and in situ testing will be required during final design to confirm rock properties and the results of stability analyses.

The dam and the thrust blocks will be founded on sound rock. This will require complete removal of all the overburden and weathered rock. Along some of the northwest trending shear zones, the weathering could be as deep as 200 feet. Extensive dental excavation may be required in these areas to form an acceptable foundation. The entire dam foundation area should be consolidation grouted to fill all the openings and cavities in rock at shallow depth.

The permeability of rock varies from 1×10^{-4} cm/sec at shallow depths to 1×10^{-6} cm/sec at depths below 175 feet. The permeability is controlled by the discontinuities in the rock and may vary widely from area to area. A grout curtain will be provided under the entire dam including the abutments and an appropriate distance beyond the dam into the abutments. A system of drain holes and drainage galleries will be included to control uplift pressures and to safely release seepage water. A double row curtain is proposed. There is little evidence of permafrost at this site; however, provision should be made for thawing during grouting should permafrost be encountered.

(b) Underground Structures

The rock conditions at the site are suitable for the construction of tunnels and underground caverns. From the geological and geotechnical viewpoint, the location and the orientation of these structures are influenced by the orientation and location of rock discontinuities.

The RQD values indicate that about 50 percent of the rock is in the good-to-excellent category, roughly 40 percent in the fair-to-good category, and the remaining percentage in the poor category. The poor quality rock is generally associated with fractures and shear zones. The major joint sets are oriented northwest (Set I) and northeast (Set II). Both sets are steeply dipping. The bedding plane strikes roughly in a NE-E direction and dips at 45 to 80° southeast. The orientation of the tunnels and the large underground caverns have been carefully selected to minimize the potential adverse effects of these rock discontinuities.

Determination of the magnitude and the orientation of the in situ stresses will not be possible until in situ testing is undertaken. Nevertheless, the tectonic setting suggests that the entire site region is in a compressional stress regime. Near valley walls, the stresses are expected to have been relieved and low horizontal stresses may exist. Considering the unconfined strength of the intact rock, overstressing problems such as rock spalling and slabing are not anticipated. The rock support requirements will depend on the size and orientation of the openings and the presence and character of the rock discontinuities intersected. For the most part, conventional rock bolt support using 3/4-inch to 1-inch-diameter bolts has been assumed to be adequate for openings less than 40 feet in span. For larger spans, in areas of poor quality rock and where rock discontinuities are known to be adversely oriented, support requirements have been determined on a case-by-case basis. In the case of large span openings (such as powerhouse cavern), special attention has been given to the potential presence of subhorizontal joints where they may intersect almost vertical joints and may create unstable blocks in the crown of the excavation. The use of shotcrete, welded wire fabric, and concrete lining will be required in poor rock quality areas. For power tunnels, provisions have also been made for concrete lining and contact/consolidation grouting.

Although rock permeabilities are generally low to moderate, intersection of rock discontinuities may lead to ground water inflow problems during construction and cause high-pore water pressures after the reservoir is flooded. Therefore, provisions have been made for grouting around tunnels and caverns, and suitably placed drain holes and drainage galleries have been provided upstream from the powerhouse and surge chamber.

The excavation of tunnels may be performed using conventional drill and blast techniques or high-production mechanical excavators. Sufficient information is not available at this time to select an optimum system. For cost estimating purposes, conventional methods have been conservatively assumed at this time. The excavation of the powerhouse cavern will be by drill and blast using a heading, side slash, and benching sequence.

The spacing between long tunnels should be 2.5 times the diameter of the largest tunnel. The spacing between the caverns should be kept so that a minimum pillar thickness of 1.5 times the span of the larger cavern is maintained.

(c) Stability of Soil and Rock Slopes

In most areas, the permanent excavation slopes will be confined within the rock, except on the left bank, where a deep buried stream exists. The slopes within overburden will depend on the nature of soil, ground water table, and the height of the slope. In general, slopes within the overburden have been assumed as 2H:1V or less below the water table and 1.5H:1V or less above the water table. A bench of appropriate width will be provided at the overburden-rock contact to accommodate any local slumping/slope failure and to intercept and dispose of ground/seepage water. Flatter slopes are required in some areas where frozen ground may become unstable because of high pore pressures during thawing or where slope height/soil conditions so dictate.

The slopes of excavations in rock have been selected in accordance with the joint dips and orientations and the shear strength of rock along discontinuities. Slopes in intact rock or where discontinuities dip into the excavated face will usually stand steeply without any structural support. Slopes paralleling the discontinuity have, whenever possible, been laid back to the same angle as the dip of the rock discontinuity or adequate rock support provided. Wherever possible, permanent cuts have been set at stable slopes without the need for rock bolts. In areas where large pore pressures could potentially develop behind the rock cuts, allowances have been made for drain holes to relieve the pore pressures. In general, a 4V:1H overall slope is considered stable. For slopes exceeding 40 feet in height, benches have been included every 40 feet. Excavation of tunnel portals includes pattern rock bolting and appropriate provision for concrete/shotcrete to reduce the risk of unstable slopes. Special details are required in areas where slopes will intersect or cross larger shear zones or otherwise unstable rock.

(d) Saddle Dam Foundation

The saddle dam on the south bank will be constructed across the buried stream. The thickness of overburden in this area reaches up to 80 feet. The bedrock below (argillite and graywacke) area is competent. The impervious core, filters, and outer shells for the saddle dam will be founded on sound rock. The prominent shear zone or fault which was found in the saddle dam foundation, together with various shear and fracture zones, has been treated by means of provisions for consolidation and curtain grouting under the core as a continuation of foundation treatment for the arch dam.

10.4 - Seismic Considerations

The seismicity of the Susitna Basin and the sources of earthquakes are discussed in Section 7 of this report. This section presents the implications of the seismicity on the design of the Devil Canyon project.

(a) Seismic Design Approach

For the earthquake engineering and design considerations, the project structures have been classified as either critical structures or noncritical structures. Critical structures include the dam and similar major structures whose failure may result in sudden and uncontrolled release of large volumes of water which may endanger property and lives downstream. The noncritical structures are those structures whose failure can be assessed as an economic or financial loss to the project in terms of lost revenue, repair, and/or replacement cost. Critical structures will be designed to safely withstand the effect of the "Safety Evaluation Earthquake" (SEE) for the site. No significant damage to these structures will be accepted under these conditions.

For design of critical structures, the effective acceleration for the SEE has been determined as 0.8 x actual SEE acceleration, together with a corresponding scaled response spectrum. The selected SEE corresponds to the "terrain" or "detection level" earthquake which has been characterized as follows:

- Magnitude:	6-1/4 to 6-1/2
- Location:	Approximately 3 km from structure
- Maximum Acceleration:	Me'., 0.55g to 0.60g
	84th percentile 0.70 g
- Peak Spectral Acceleration:	Mean 1.37g to 1.50g
	84th percentile 1.77g

The response spectra for this event are shown in Figure 10.7A. The effective peak acceleration for design of structures is then:

- Design a max = 0.8 x 0.70g = 0.56g
  (Sa) max = 0.56g x 2.5 = 1.40g

The design of non-critical structures for earthquake conditions will be undertaken using conventional Uniform Building Code recommendations.

(b) Safety Evaluation Earthquake

The design of the Devil Canyon arch dam and other critical structures has been undertaken using response spectrum analysis. Although the "terrain" earthquake would result in more severe ground motions, the duration of these motions is relatively short. However, the method of analysis used for the Devil Canyon Dam does not take duration into account. The most likely source of strong ground shaking at the Devil Canyon site is, in fact, the Benioff Zone. The estimated mean peak response spectrum for the SEE is presented in Figure 10.7, along with the 84th percentile response spectrum. A maximum horizontal acceleration level for the 84th percentile response spectrum for the Benioff event is approximately 0.47g.

10.: - Selection of Reservoir Level

The selected normal maximum operating level at Devil Canyon Dam is Elevation 1455. Studies by the USBR and COE on 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.

The detailed studies of reservoir level at Watana (Section 9) indicated little change in benefit-cost ratio over a 100-foot range of reservoir level at the upper limit. Maximization of hydroelectric energy production at the site was found to be an important objective which weighed heavily in the selection of reservoir level at Watana. Although a detailed determination has not been undertaken, the same is likely to be true at Devil Canyon.

Although significantly lower reservoir levels at Devil Canyon would lead to lower dam costs, it is clearly evident that 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.

10.6 - Selection of Installed Capacity

The methodology used for the preliminary selection of installed capacity at Watana and Devil Canyon is described in Section 9.6.

The decision to operate Devil Canyon essentially as a run-of-river plant with maximum utilization of available flows from Watana 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 Devil Canyon will have a significant impact on 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 res lts of reservoir operation computer simulation analysis (Section 9.6), with the station operating at 100 percent load factor. 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 power flow releases at Watana will also be used at Devil Canyon.

The total capacity required at Devil Canyon in a wet year, excluding standby and spinning reserve capacity, is summarized below. As discussed in Section 9.6, the capacity shown is based on the Battelle 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 feasibility design purposes. This will provide some margin of standby for 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.

For feasibility design purposes, the power facilities at Devil Canyon have been developed for 4 units at 150 MW. This arrangement will provide for efficient station operation at part load. Consideration of phasing of the capacity installation of the machines may be desirable as the system demand increases. However, the uncertainty of load forecasts this fan into the future, and the additional contractual costs of mobilization for equipment installation are such that for study purposes at this time it has been assumed that all units will be commissioned by 2002.

The Devil Canyon reservoir will normally be full in December; hence, any forced outage will result in spilling and a loss of available energy. The units have been rated to deliver 150 MW at minimum December drawdown level; 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.

10.7 - 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 (Section 9). An emergency spillway with an erodible fuse plug will also be provided to handle larger discharges up to the probable maximum flood. As discussed in Section 8 and elsewhere, the development plan envisages completion of the Watana project prior to construction at Devil Canyon. Accordingly, the inflow flood peaks will be significantly less at Devil Canyon because of routing through the Watana reservoir. Spillway floods as calculated in Section 7.2 are:

Flood	Inflow Peak (cfs)
Probable Maximum	366,000
1:10,000 year	165,000

The restrictions with respect to nitrogen supersaturation of downstream flows discussed in Section 9 for Watana also will apply to Devil Canyon, and discharges of nitrogen-supersaturated water from Devil Canyon will be limited to a recurrence period of not less than 1:50 years.

10.8 - 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 these feasibility studies to evaluate the relative merits of the following types of structures at the same location:

- Concrete thin arch;
- Concrete gravity arch; and
- Fill embankment.

(a) Comparison of Embankment and Concrete Type Dams

This analysis was based on the concrete arch and concrete gravity arch schemes developed by the COE in 1975 and 1978, together with a rockfill dam alternative developed as part of the current study program. The results of the analysis indicated a trend in favor of the concrete arch dam alternative when compared to the gravity or rockfill dam alternatives. The assessment showed that a gravity dam in the narrow gorge would tend to behave similarly to an arch dam but would not have the flexibility of such a structure. The technical feasibility of a gravity dam was therefore questionable particularly under severe seismic shaking conditions. This type of dam also tended to be more expensive and was, therefore, not considered further.

Consideration of a central core rockfill dam at Devil Canyon indicated relatively small cost differences from a conservative arch dam, significantly thicker in cross section than the finally selected design. Furthermore, no information was available to indicate that impervious core material could be found for such a dam in the necessary quantities and within a reasonable distance. The rockfill dam was accordingly dropped from further consideration. Details of this evaluation are presented in Appendix D2.

Neither of the concrete arch dam layouts are 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 of where the river enters Devil Canyon 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.

(i) <u>Thick Arch Dam</u>

The main concrete dam will 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 will 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 will be 1460. The center portion of the dam will be founded on a massive mass concrete pad constructed in the excavated river bed. This central section will incorporate a service 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 will terminate in thrust blocks high on the abutments. The left abutment thrust block will incorporate an emergency gated control spillway structure which will discharge into a rock channel running well downstream and terminating at a high level in the river valley.

Beyond the control structure and thrust block, a low-lying saddle on the left abutment will be closed by means of a rockfill dike founded on bedrock. The powerhouse will house 4 x 150 MW units and will be located underground within the right abutment. The multi-level intake will be constructed integrally with the dam and connected to the powerhouse by vertical steel-lined penstocks.

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

(ii) Thin Arch Dam

The main dam will 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 will be 1460. The center section will be founded on a concrete pad, and the extreme upper portion of the dam will terminate in concrete thrust blocks located on the abutments.

The main service spillway will be located on the right abutment and will consist of a conventional gated control structure discharging down a concrete-lined chute terminating in a flip bucket. The bucket will 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 will be supplemented by orifice type spillways located high in the center portion of the dam which will 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, will be located beyond the saddle dam on the left abutment.

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

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

(iii) Comparison of Arch Dam Types

Sand and gravel for concrete aggregates are believed to be available in sufficient quantities within economic distance from the dam as discussed in Sections 10.2 and 10.3. 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 a moderate amount of screening and washing.

The bedrock geology of the site is discussed in Sections 10.2 and 10.3. At this stage it appears that there are no geological or geotechnical concerns that would preclude either of the dam types from consideration.

The thick arch dam will allow for the incorporation of a main spillway chute on the downstream face of the dam discharging into a spillway located deep within the present riverbed. This spillway will be able to pass routed floods with a return frequency of less than 1:10,000 years. For the thin arch and rockfill alternatives, the equivalent discharge capacity will be provided separately through the abutments.

Under hydrostatic and temperature loadings, stresses within the thick arch dam will be generally lower than for the thin arch alternative. However, finite element analysis has shown that the additional mass of the dam under seismic loadings will 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 will 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.

The thick arch arrangement did not appear to have any outstanding merits 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.

10.9 - Diversion Scheme Alternatives

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

(a) 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.

(b) Design Flood for Diversion

The recurrence interval of the design flood for diversion was established in the same manner as for Watana (see Section 9). 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, is presented below:

Average annual flow:	7,860 cfs
Maximum average monthly flow:	23,100 cfs (June)
Minimum average monthly flow:	890 cfs (March)
Design flood inflow (1:25 years routed through Watana reservoir):	37,800 cfs

(c) 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 relatively pervious alluvium material will be achieved by means of a grouted zone. The nature of the alluvium led to the selection of a grouted zone rather than a slurry wall.

The selected cofferdam sections are described in more detail in Section 13.

(d) 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 concretelined tunnels, design velocities of approximately 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. Using a maximum permissible velocity of 10 ft/s, four unlined tunnels, each with an equivalent circular diameter of 35 feet, would be required to pass the design flow. Alternatively, a design velocity of 50 ft/s would theoretically permit the use of one concrete-lined tunnel with an equivalent diameter of 30 feet. 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 (Section 9) indicated that cofferdam closure problems associated with free-flow tunnels would more than offset their other advantages.

Pressure tunnels are designed to flow full and, accordingly, must withstand internal pressure. The most widely used type of pressure tunnel for diversion has the crown of the outlet portal submerged during all flow conditions. The tunnel cross-section used for this evaluation was a modified horseshoe or "D" shaped configuration. The area of this type of section is

13.7 percent greater than for a circular tunnel with the same diameter of span. The "D"-shaped tunnel offers advantages in terms of ease of construction and scheduling for the spans envisaged at Devil Canyon.

(e) 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.

Capital costs were developed for these single 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 intake and outlet gate structures 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 assumed to be 2,000 feet, reflecting a left bank location.

Rating curves for the single-pressure tunnel alternatives are presented in Figure 10.8. The relationship between capital cost and crest elevation for the upstream cofferdam is shown in Figure 10.9. The capital cost for various tunnel diameters is given in Figure 10.10.

The results of the optimization study are presented in Figure 10.11 and indicate the following optimum solutions for each alternative.

Tunnel Diameter	Cofferdam Elevation	Total Cost
25 feet	945 feet	$8,000,000
30 feet	945 feet	$6,600,000
35 feet	945 feet	$7,100,000

The selection of the diversion scheme was based on economics, a single, 30-foot-diameter pressure tunnel being selected. An upstream cofferdam 60 feet high, with a crest elevation of 945, was carried forward as part of the selected general arrangement.

The various components of the selected diversion scheme are described in Section 13.

10.10 - Spillway Alternatives

As discussed in Section 10.7, the project has been designed to safely pass floods with the following return frequencies:

10.10 - Spillway Alternatives

As discussed in Section 10.7, the project has been designed to safely pass floods with the following return frequencies:

Flood	Frequency	Discharge (cfs)
Spillway Design Flood	1:10,000 years	135,000
Probable Maximum Flood	--	270,000

A number of alternatives were considered singly and in combination for Devil Canyon spillway facilities. These included gatd overflows or orifices in the main dam discharging into a plunge pool downstream, right and left bank chute or tunnel spillways in the flip buckets or stilling basins for energy dissipation, and emergency open channel spillways. As described for Watana in Section 9, the selection of spillway facilities greatly influenced and was influenced by the general arrangement of the major structures. In general, the main spillway facilities will discharge the design flood through a gated spillway control structure with energy dissipation either by a flip bucket which directs the spillway discharge in a free fall jet into a plunge pool in the river or by a stilling basin which dissipates the energy in a hydraulic jump. In addition, similar restrictions apply with respect to limiting nitrogen supersaturation in the spillway discharges. The various spillway arrangements developed in accordance with these considerations are discussed in Sections 10.13 and 10.14.

10.11 - Power Facilities Alternatives

The selection of the optimum arrangements for the power facilities involved consideration of the same factors as described in Section 9.11 for Watana. The selection of the installed capacity of 600 MW at Devil Canyon is described in Section 10.6.

(a) 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.

(b) 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.

(c) 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 9.11.

The selection of the number and rating of individual units is discussed in detail in Section 10.6. The four units will each be rated to deliver 150 MW at full gate opening at minimum reservoir level in December (the peak month). The best efficiency unit output at rated head (575 feet) is 164 MW.

(d) Transformers

Transformer selection is similar to the procedure for Watana as discussed in Section 9.11.

The arrangement of the transformers at Devil Canyon is described in detail in Section 13.

(e) Power Intake and Water Passages

For flexibility and security of operation, individual penstocks are provided to each of the four units. As discussed in Section 9.11 for Watana, it was found that there is no significant cost advantage in using two larger diameter penstocks with bifurcation at the powerhouse.

A single tailrace tunnel has been assumed 6,800 feet in length, to develop a further 30 feet of head downstream from the dam. Detailed design may indicate this should be changed to two smaller tailrace tunnels for improved station security; the extra cost involved would be insignificant. The surge chamber design for two tailrace tunnels would be relatively unchanged.

The overall length of the intake structure is governed by the selected penstock diameter and the minimum penstock spacing. Detailed studies were carried out to determine the optimum diameter of the penstocks and the tailrace tunnel, in a similar manner to that described for Watana in Section 9.11.

(f) Environmental Constraints

In addition to potential nitrogen-saturation problems caused by spillway operation, as discussed in Section 10.10, the major impacts of the Devil Canyon power facilities development are:

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

Temperature modeling has indicated that 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 normal reservoir operating level (El 1455).

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

10.12 - General Arrangement Selection

The approach to selection of a general arrangement for Devil Canyon was a similar but simplified version of that used for Watana, as described in Section 9.

(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 initially, four schemes were 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 is presented in Table 10.2. Subsequent to selection of the preferred Devil Canyon scheme, the information was refined and updated as part of the on-going study program. The description of the Devil Canyon project presented in Section 13 reflects the most recent design data for the project.

10.13 - 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. As discussed above, 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.

(a) 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 a mass concrete plug, constructed 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 symetrical and contributes to a more uniform stress distribution.

(i) Scheme DC 1 (See Plate 23)

In this scheme, diversion facilities comprise upstream and downstream earthfill and rockfill cofferdams and two 24-foot-diameter tunnels beneath the left abutment (Section 10.9).

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 ski-jump 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 33,000 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 in to 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. Thus, as 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 (see Section 10.3), 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.

(ii) <u>Scheme DC 2</u> (See Plate 24)

The layout is generally similar to Scheme DC 1 except that the chute spillway is located on the left side of the canyon. The concrete-lined chute terminates in a ski-jump 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 DC 1, 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.

(iii) Scheme DC 3 (See Plate 25)

The layout is similar to Scheme DC 1 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 DC 1.

(iv) Scheme DC 4 (See Plate 26)

The dam, power facilities, and saddle dam for this scheme are the same as those for Scheme DC 1. 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. This arrangement forms the service spillway with central orifices in the dam and the left bank rock channel and fuse plug forming the auxiliary and emergency spillways, respectively, as in the alternative schemes.

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

(b) Comparison of Alternatives

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

As can be seen from a comparison of the costs in Table 10.___, the flip bucket spillways are substantially less costly to construct than the stilling-basin type of Scheme DC 4. The left side spillway of Scheme DC 2 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 be a considerable problem causing undermining of the canyon sides and their subsequent instability, together with the possibility of a deposition of material downstream 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 spillways schemes, based on either an open chute or a tunnel, take advantage of a downstream bend in the river to eject discharges parallel to the course of the river. This will reduce the effects of erosion but could still present a problem, as can be seen from the outline of the estimated maximum possible scour hole which would occur over a period of time.

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 all spillways, could particularly cause troubles in the tunnel with the possibility of spiraling flows and severe cavitation.

The stilling basin type spillway of Scheme DC 4 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, introducing air into the water/concrete contact area at offsets along the chute invert. There is, however, little precedent for stilling basin operation at heads of over 550 feet; and even where floods of much less than the design capacity have been discharged, severe damage has occurred.

(c) Selection of Final Scheme

The chute and flip bucket spillways of Schemes DC 1 and DC 2 could generate downstream erosion problems which could, in the case of Scheme DC 2, require considerable maintenance costs and cause reduced efficiency in operation of the project at a future date. Scheme DC 3 causes hydraulic problems and cavitation could be severe. There is no cost advantage in this type of spillway over the open chute. In Scheme DC 4, the operating characteristics of a high head stilling basin are little known, and there are few examples of successful operation.

All spillways operating at the required heads and discharges will eventually cause some erosion. For all schemes, use of auxiliary release facilities in the dam to handle floods up to 1:50-year frequency is considered a reasonable approach to limit erosion and nitrogen supersaturation problems. The cost of the flip bucket type spillway in the scheme is considerably less than that of the stilling basin in Scheme DC 4. The latter offers no relative operational advantage; therefore, Scheme DC 1 has been selected for further study.

10.14 - Final Review

The layout selected in Section 10.13 was further developed in accordance with updated engineering studies and criteria. The major change compared to Scheme DC 1 is in the central spillway configuration, but other modifications that were introduced are described below.

The revised layout is shown on Plate 27. A description of the structures is as follows.

(a) 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 Elevation to 1463 with the concrete parapet wall crest at Elevation 1466. The saddle dam was raised to Elevation 1472.

(b) Spillways and Outlet Facilities

To alleviate the potential for nitrogen supersaturation problems, it was necessary to restrict supersaturated flow to an average recurrence interval of not less than 50 years. In order to pass floods of greater freqency, an alternative type of discharge facility was required.

In addition, it was considered probable that frequent and costly maintenance would be required in the concrete-lined plunge pool beneath the central orifice spillways and just downstream from the dam. This is a critical area because of the proximity to the dam.

These two considerations led to the replacement of orifice spillways by outlet facilities incorporating five fixed-cone valves, with a diameter of 108 inches, capable of passing a design flow of 45,000 cfs.

The chute spillway and flip bucket are located on the right bank, as in Scheme DC 1; however, the chute length was decreased and the elevation of the flip bucket raised compared to Scheme DC 1.

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

(c) Diversion

The previous twin diversion tunnels were replaced by a single-tunnel scheme. This was determined to provide all necessary security but will be slightly less expensive than the two-tunnel alternative (see Section 10.9).

(d) 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.

LIST OF REFERENCES

(1) Acres American Incorporated, Report on 1980-81 Geotechnical Investigations, February 1982.

(2) Woodward-Clyde Consultants, Interim Report of Seismic Studies for the Susitna Hydroelectric Project, December 1980.

(3) Woodward-Clyde Consultants, Final Report of Seismic Studies for the Susitna Hydroelectric Project, February 1982.

TABLE 10.2: 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:	1,455 feet
Reservoir minimum operating level:	1,430 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:	1,455 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 10.2: (Continued)

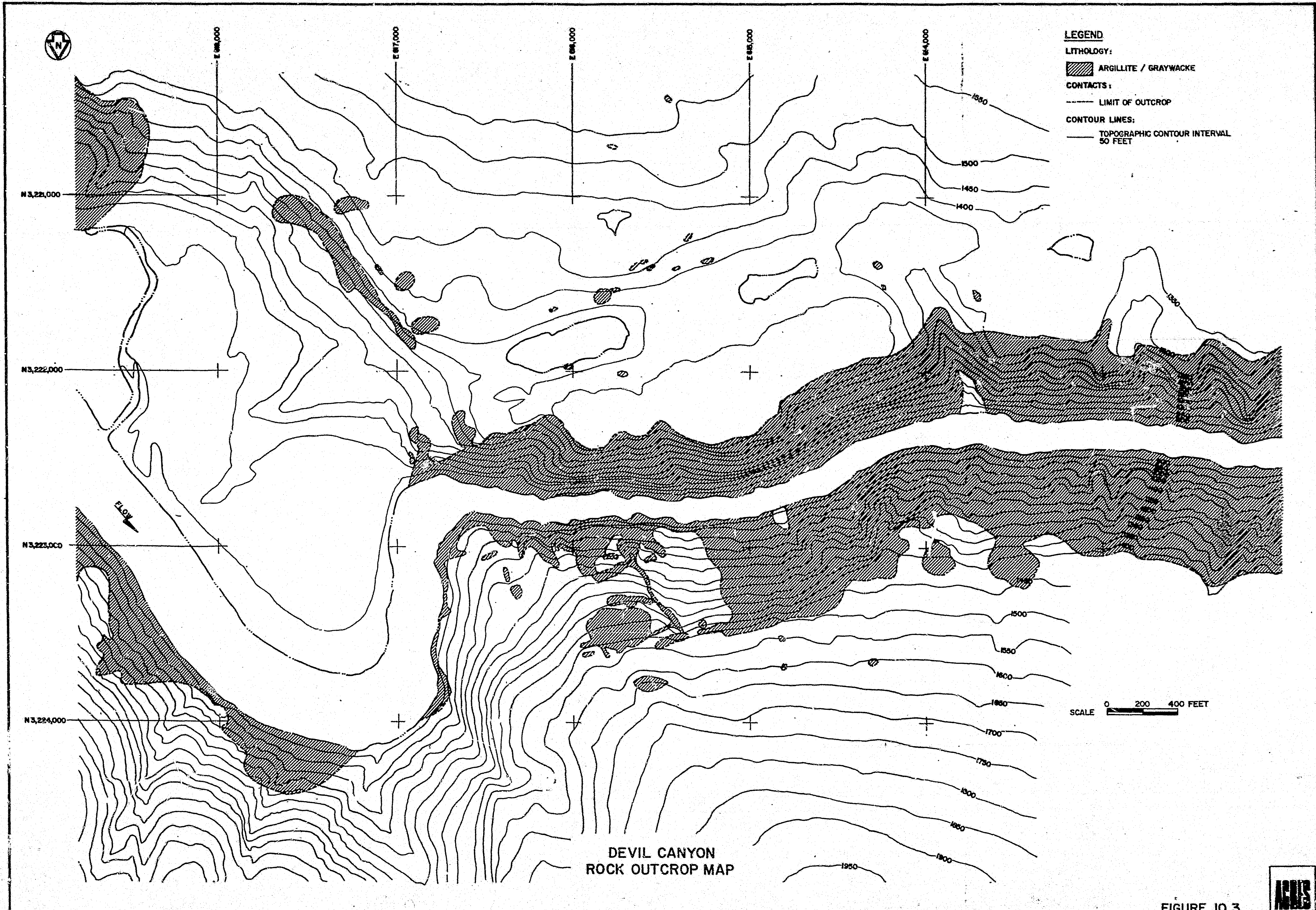
Power Intake

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:	___ MVA
Power factor:	0.9

TABLE 10.3: SUMMARY OF COMPARATIVE COST ESTIMATES

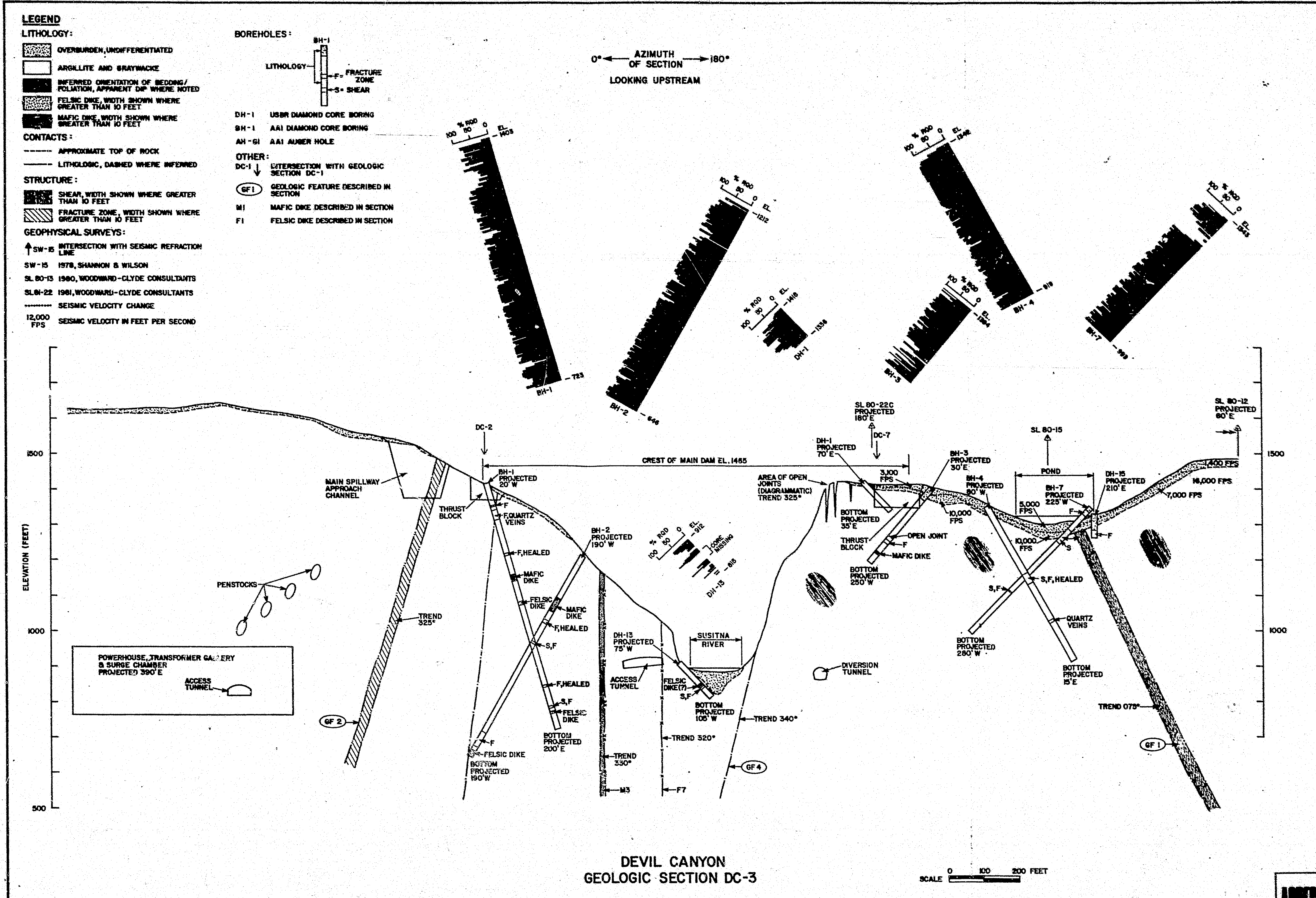
PRELIMINARY REVIEW OF ALTERNATIVE ARRANGEMENTS
(January 1982 $ Millions)

Item	DC 1	DC 2	DC 3	DC 4
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
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



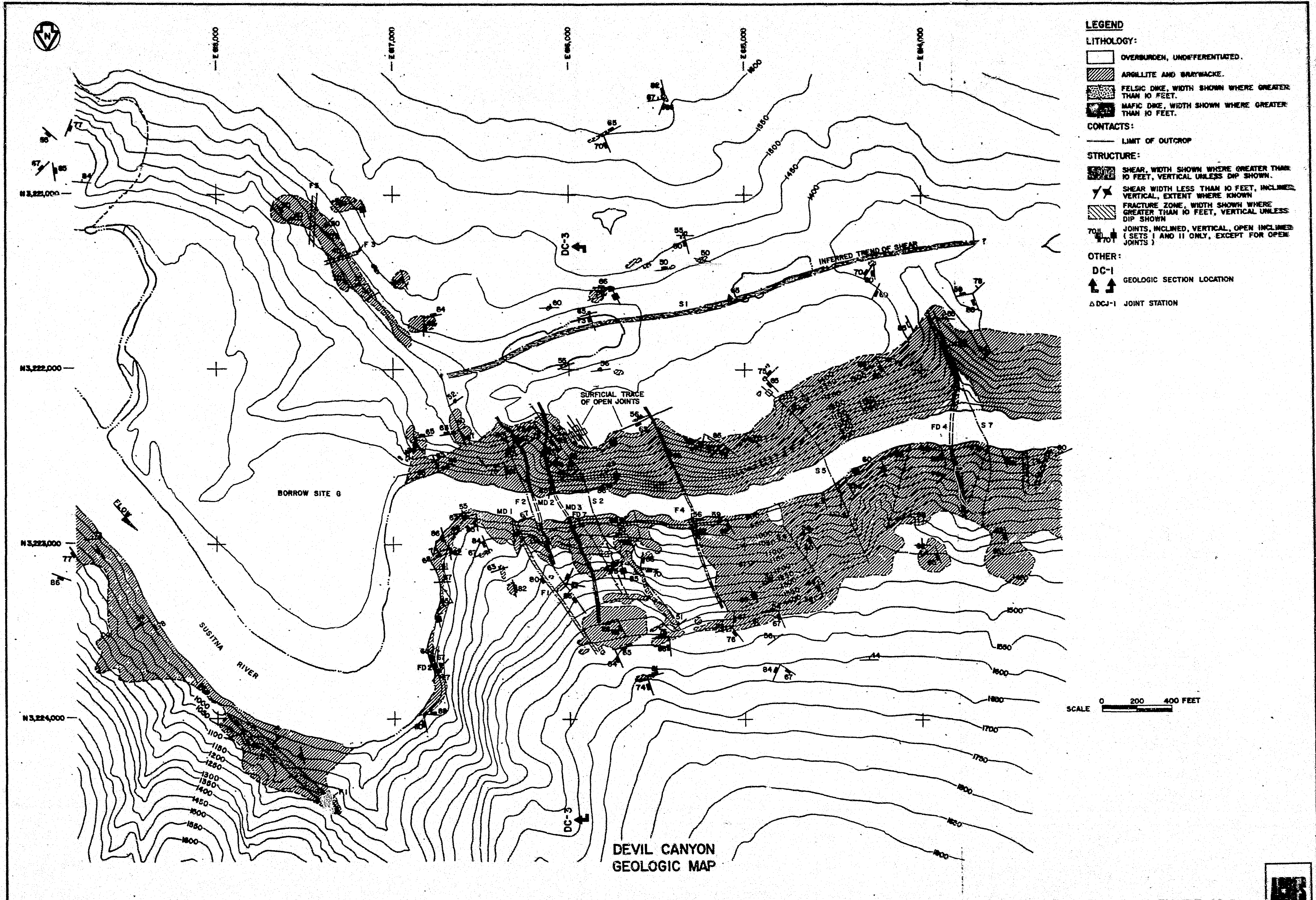
LEGEND
LITHOLOGY:
ARGILLITE / GRAYWACKE
CONTACTS:
LIMIT OF OUTCROP
CONTOUR LINES:
TOPOGRAPHIC CONTOUR INTERVAL 50 FEET
E 68,000
E 67,000
E 66,000
E 65,000
E 64,000
N 3,221,000
N3,222,000
N3,223,000
N3,224,000
FLOW
SCALE 0 200 400 FEET
DEVIL CANYON
ROCK OUTCROP MAP

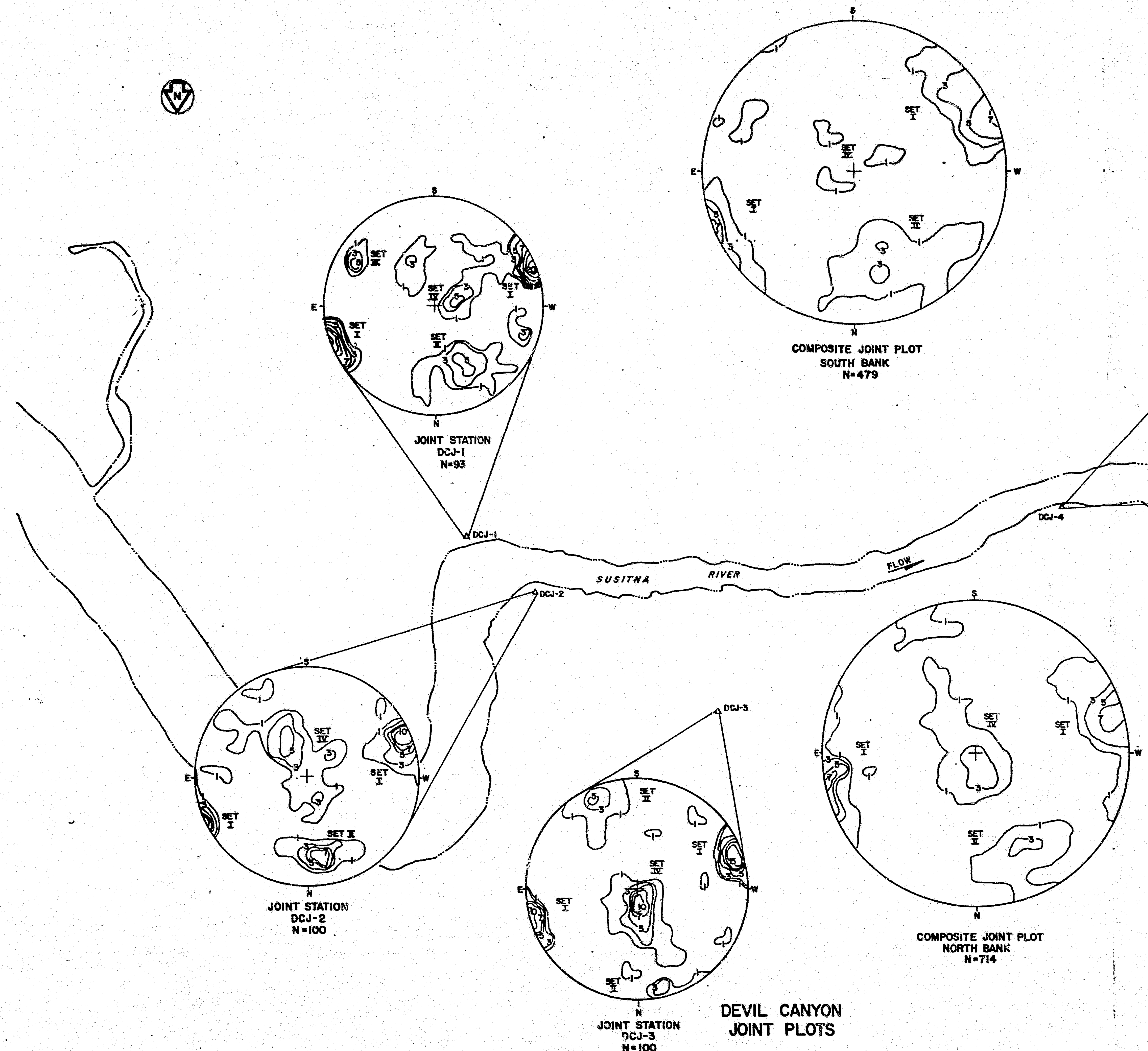
FIGURE 10.3



DEVIL CANYON
GEOLOGIC SECTION DC-3

FIGURE 10.4


LEGEND
LITHOLOGY:
OVERBURDEN, UNDIFFERENTIATED.
ARGILLITE AND GRAYWACKE.
FELSIC DIKE, WIDTH SHOWN WHERE GREATER THAN 10 FEET.
MAFIC DIKE, WIDTH SHOWN WHERE GREATER THAN 10 FEET.
CONTACTS:
LIMIT OF OUTCROP
STRUCTURE:
SHEAR, WIDTH SHOWN WHERE GREATER THAN 10 FEET, VERTICAL UNLESS DIP SHOWN.
SHEAR WIDTH LESS THAN 10 FEET, INCLINED, VERTICAL, EXTENT WHERE KNOWN
FRACTURE ZONE, WIDTH SHOWN WHERE GREATER THAN 10 FEET, VERTICAL UNLESS DIP SHOWN
JOINTS, INCLINED, VERTICAL, OPEN INCLINED (SETS I AND II ONLY, EXCEPT FOR OPEN JOINTS)
OTHER:
DC-1
GEOLOGIC SECTION LOCATION
DCJ-1 JOINT STATION
SCALE 0 200 400 FEET
INFERRED TREND OF SHEAR
SURFICIAL TRACE OF OPEN JOINTS
BORROW SITE G
FLOW
SUSITNA RIVER
DC-3
N 3,221,000
N 3,222,000
N 3,223,000
N 3,224,000
E 88,000
E 87,000
E 86,000
E 85,000
E 84,000
DEVIL CANYON GEOLOGIC MAP
FIGURE 10.5

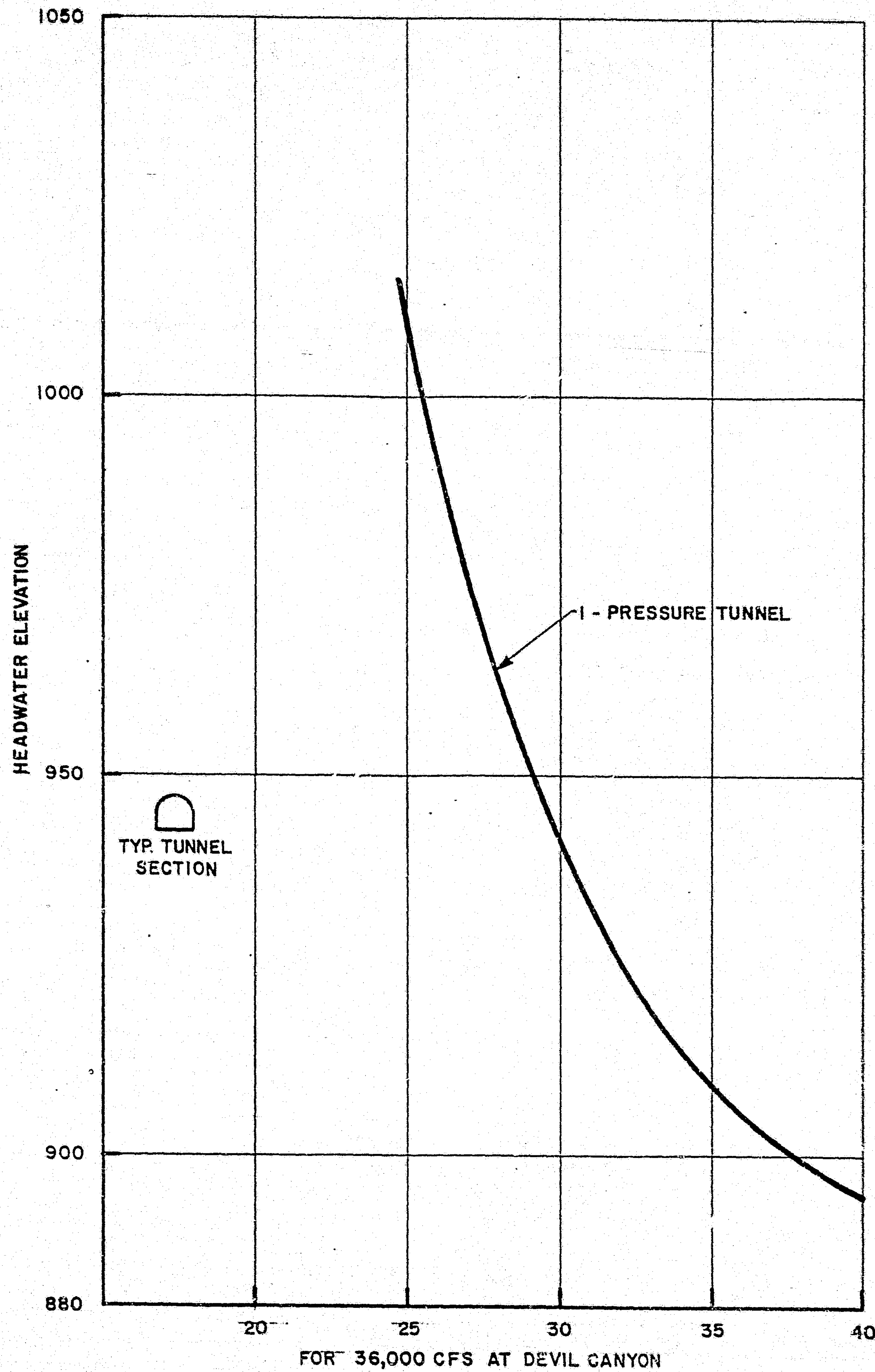


NOTES

1. CONTOURS ARE PERCENT OF JOINTS PER 1% OF AREA. CONTOUR INTERVAL - 1,3,5,7,10,15, & 20%.
2. N EQUALS NUMBER OF DATA POINTS.
3. COMPOSITE PLOTS INCORPORATE ALL JOINT DATA. JOINT STATION PLOTS CONTAIN DATA FROM SPECIFIC JOINT STATIONS.
4. FOR JOINT PLOTTING METHOD SEE

DEVIL CANYON
JOINT PLOTS

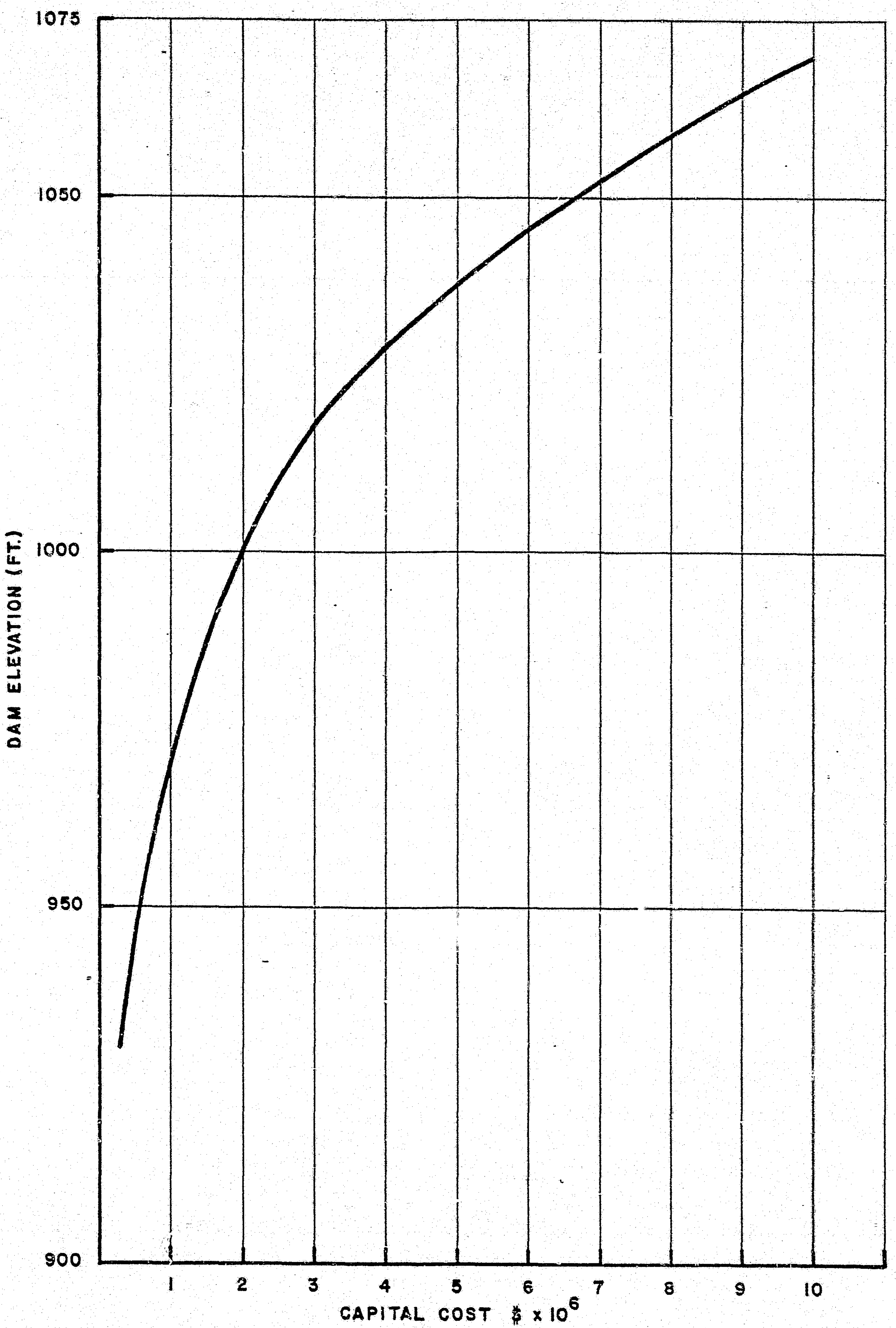
FIGURE 10.6



DEVIL CANYON DIVERSION
HEADWATER EL./ TUNNEL DIAMETER

FIGURE 10.8

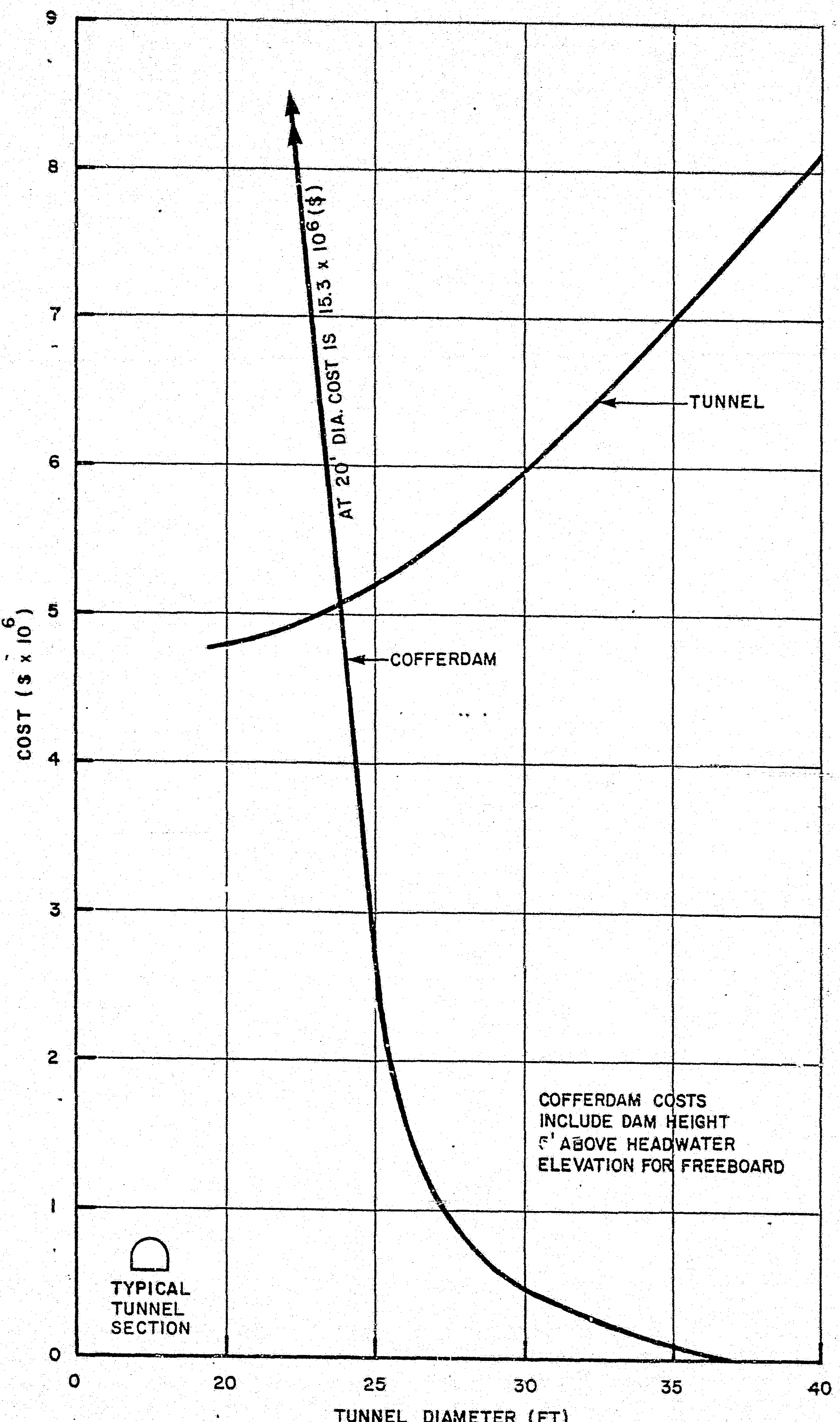
ACRES



DEVIL CANYON DIVERSION
UPSTREAM COFFERDAM COSTS

FIGURE 10.9

ACRES

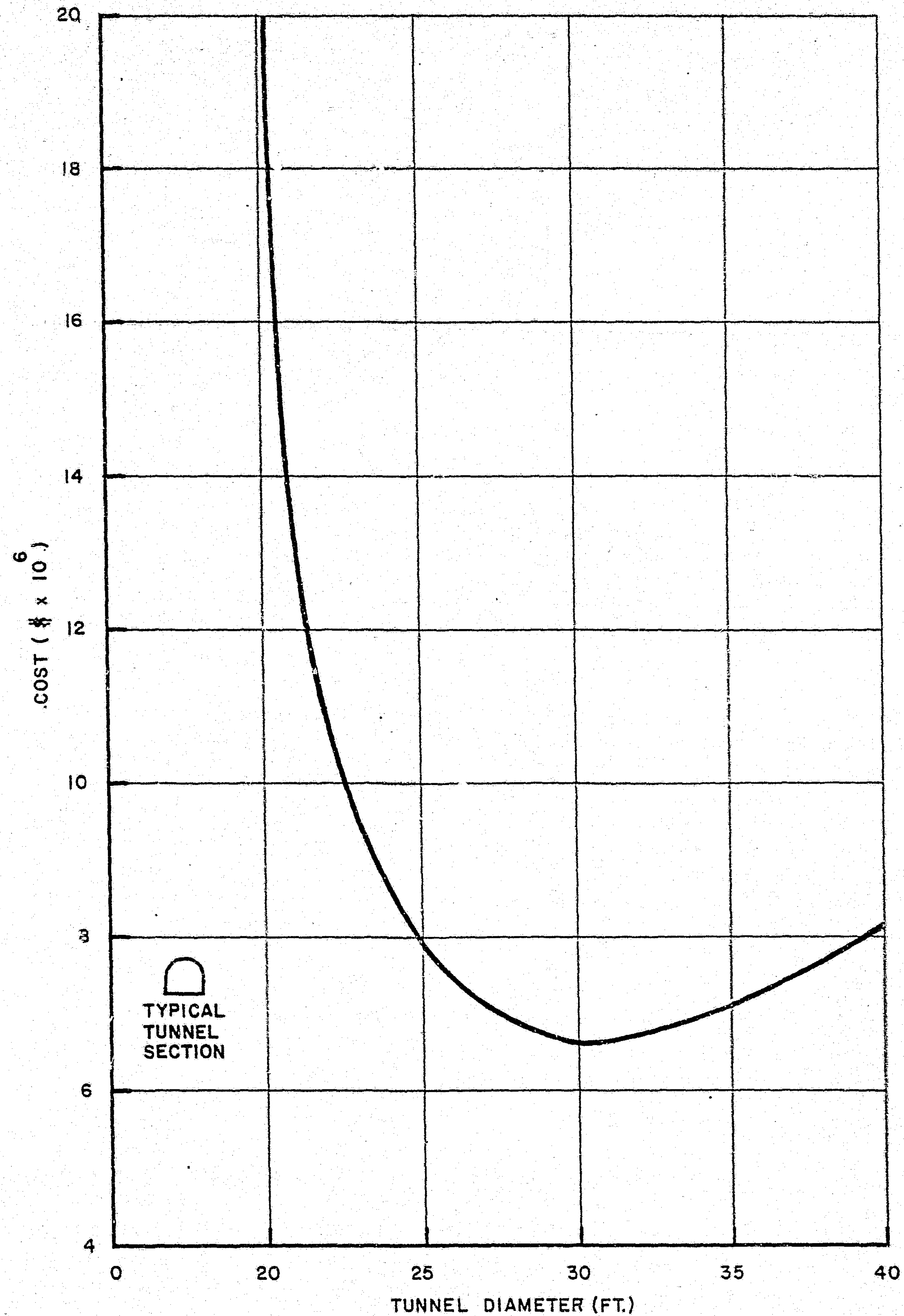


DEVIL CANYON DIVERSION

TUNNEL & COFFERDAM COST / TUNNEL DIAMETER

FIGURE 10.10

ACRES



DEVIL CANYON DIVERSION
TOTAL COST / TUNNEL DIAMETER

FIGURE 10.11

ACRES

11 - SELECTION OF ACCESS PLAN

This section describes the process of formulation and selection of an access plan for the Susitna Hydroelectric Project. The methodology for comparison of alternative plans is outlined, and an evaluation of each basic plan is presented, considering schedule, costs, and biological and social impacts. The selected plan is described in detail, and recommendations for measures to reduce impacts are presented.

Engineering studies conducted on the alternative routes consisted of development of design criteria, layouts of the alternative routes, preliminary field investigations, cost estimates of constructing the alternative routes and logistics costs in transporting supplies and materials to the dam sites. Environmental studies included identification and evaluation of biological impacts for each of the alternative routes. The environmental studies included field investigations and assessments for all the alternative routes. Social or socioeconomic studies included a public participation program among the various studies. Public concerns and preferences, particularly those of the sector that would be impacted the most directly, were solicited and fully considered in the evaluation.

The evaluation of the alternative plans included development of selection criteria, comparisons of the alternative plans, identification of conflicts among the alternatives in the evaluation criteria, comparison of the conflicts in the criteria, and the tradeoffs made in the evaluation.

11.1 - Background

(a) Existing Access Facilities

As discussed elsewhere, the proposed Devil Canyon and Watana sites are located approximately 115 miles northeast of Anchorage and 140 miles south of Fairbanks. The Alaska Railroad, which links Anchorage and Fairbanks, passes within 12 miles of the Devil Canyon site at Gold Creek. The George Parks Highway (Route 3) parallels the Alaska Railroad for much of its route, although between the communities of Sunshine and Hurricane, the Highway is routed to the west of the Railroad, so that Gold Creek is situated approximately 16 miles south of the intersection of the railroad and highway. A portion of the highway between Lane and Hurricane, known as the Parks Highway, passes through Denali State Park. At Cantwell 51 miles north of Gold Creek, the Denali Highway (Route 8) leads easterly approximately 116 miles to Paxson, intersecting the Richardson Highway at this point.

To the south, the Glenn Highway (Route 1) provides the main access to Glennallen and intersects the Richardson Highway which leads south to Valdez.

(b) Modifications to Plan of Study

The original POS proposed that a single route would be selected by May 1981 to be followed by detailed environmental investigations of this route.

Early in the study, three main access corridors were developed. Consideration of these plans on the basis of information received, comments and concerns from various state agencies and a recommendation from the Susitna Steering Committee, led to a decision to assess three alternative routes in more detail throughout 1981 and recommend a selected route later in the year. Accordingly, this assessment included environmental studies, engineering studies, aerial photography, drilling and geologic mapping of all three alternatives rather than the single route initially envisaged.

11.2 - Objectives

The finally selected access plan must allow for the efficient and timely undertaking of construction and maintenance activities in order that the Susitna Hydroelectric Development can be completed and electric power be reliably and continuously provided to the Railbelt area of Alaska.

In meeting this basic objective, several specific objectives were developed as a basis for evaluation of the alternative access routes. These objectives are:

- To allow the construction of the Susitna project to proceed on a schedule that would supply the necessary power to the Railbelt Area of Alaska when needed;

- To minimize cost including construction costs of the access facilities themselves, logistics costs for support of construction activities as well as the logistics of subsequent operation of the completed project;

- To allow for ease of operation and maintenance to ensure reliability in the power supply;

- To minimize adverse biological impacts;

- To accommodate the preferences of local communities; and

- To accommodate the preferences of Native landowners;

11.3 - Approach

The approach utilized to arrive at an access recommendation was an adaptation of the generic plan formulation and selection methodology described elsewhere in the report. The methodology as specifically applied to selection of the access plan is presented graphically in Figure 11.1.

To aid in understanding the selection process and the various studies conducted, the following definitions are provided:

(a) Corridor

A strip of land generally 2 miles or greater in width leading between two points or areas.

(b) Route

A strip of land generally 1/2 mile or less in width, leading between two points.

(c) Segment

Portions of a route which when combined constitute one alternative route between two points.

(d) Alternative Route

One of several routes which will be evaluated between two points.

(e) Plan

An access plan which will involve one or more or a combination of existing and new alternative routes. The plan will also define the logistics involved in the transportation of supplies and materials.

11.4 - Corridor Selection and Evaluation

The first step in the selection process involved identification of the three general corridors shown on Plate 28 and described below:

Corridor	Description
1	From the Parks Highway to the Watana site via the north side of the Susitna River
2	From the Parks Highway to the Watana site via the south side of the Susitna River
3	From the Denali Highway to the Watana site.

These corridors were selected based upon the use of existing transportation facilities within reasonable proximity to the Watana and Devil Canyon sites.

A general environmental analysis was undertaken for each corridor. The major environmental constraints identified within each corridor are potential impacts on the following:

(a) Corridor 1

- Fishery resources in the Susitna and Indian Rivers;
- Cliff-nesting raptors near Portage Creek and Devil Canyon;
- Furbearer habitat near Portage Creek and High Lake;
- Moose habitat on the Susitna River; and
- Caribou habitat between Devil Creek and Deadman Creek.

(b) Corridor 2

- Fishery resources in the Susitna and Indian Rivers;
- Cliff-nesting raptors near south side of the Susitna River;
- Waterfowl habitat in the Stephan Lake-Fog Lake areas; and
- Furbearer habitat in the Stephan Lake-Fog Lake areas.

(c) Corridor 3

- Caribou calving area near Butte Lake;
- Furbearer habitat; and
- Some waterfowl habitat.

In addition, increased access will cause various impacts which are common to all corridors. Archaeological resources could pose a constraint; at this time, the locations of such resources that may exist are unknown.

Finally, socioeconomic impacts will vary both in magnitude and areas of concentration, depending upon which access route or combination of access routes is selected, and whether a road or railroad is used. With the socioeconomic assessment of access schemes, there is more concern with the origin and type of access than with the actual route, because these will affect communities throughout the south-central part of the State.

With a road from the Parks Highway to the damsites (Corridors 1 and 2), effects generally would be concentrated on the western side of the project area. An easily accessible road corridor would provide for transportation of construction materials, equipment, and labor as well as post-construction uses of the upper Susitna basin (such as recreation). The impact of a railroad from the same side would likewise be concentrated on the western side. However, in every socioeconomic category, impacts would be the same or less than with the road. The single exception would be in rail industry activities, which would experience major changes.

With a road constructed from the Denali Highway to the damsites (Corridor 3), impacts along the Parks Highway-Alaska Railroad corridor would depend upon whether materials were to be shipped by road or rail to Cantwell before being transported along the Denali Highway to the access road. Impacts would occur in the Cantwell area, however, regardless of transportation mode.

11.5 - Route Selection and Evaluation

Following identification of three major corridors, a number of access routes were selected and evaluated based on engineering and economic criteria. The selected routes were then modified on the basis of an environmental analysis.

(a) Design Criteria

Construction of the Susitna project will require a dependable safe and efficient access route suitable for transporting personnel, consumable supplies and large pieces of equipment for an extended period, in adverse weather conditions.

The preliminary design criteria adopted for access road and rail alternatives were selected on the basis of similar facilities in remote projects of this nature. Basic parameters were as follows:

	Access Road	Railroad
Maximum Grade	6 percent	2.5 percent
Maximum Curvature	5 degrees	10 degrees
Design Loading		
- During Construction	80k per axle & 200k total	not applicable
- After Construction	HS-20	E-50

Following corridor definition, various segments that met engineering criteria were mapped. These segments were then jointed to form alternative routes which were compared on the basis of:

- overall length;
- average grade per mile;
- average deflection per mile;

(b) Economic Criteria

For the early stages of corridor and route selection, the alternatives were compared on the basis of total centerline length of route, with minor adjustments for average grade and curvature. Preliminary capital costs for construction were estimated at $1,250,000 per mile.

(c) Results

A total of 16 segments, combined into 30 routes were identified within the three corridors. The alternatives identified as being most favorable in terms of overall length, grade and alignment are as follows:

	Corridor 1	Corridor 2	Corridor 3	Corridor 2
	Parks Highway to Watana north side	Parks Highway to Watana south side	Denali Highway to Watana	Parks Highway to Watana south side
Type	Road	Road	Road	Rail
Overall length	64.9 miles	66.5 miles	39.1 miles	58.0 miles
Average Grade	2.4 percent	2.2 percent	1.3 percent	0.5 percent
Deflection per Mile	7° 06'±	4° 50°±	1° 30'±	5° 11'±

(d) Environmental Influences on Alternative Routes

After the engineering and economic assessment identified the 3 roads and 1 rail route described above, an initial screening was made which resulted in several refinements to the alternative routes under consideration. A major refinement involved the deletion of a large portion of the road access corridor to the Parks Highway on the north side of the river (Corridor 1). The segment connecting the highway and the Devil Canyon site routed around Portage Creek was deleted mainly on the basis of potentially severe environmental impacts on anadromous fish, furbearers, and raptors. The topography in the Portage Creek area is furthermore such that the overall length of road necessary to meet the established criteria was excessive. In addition, the construction of the segment would be extremely difficult due to the predominance of steep sidehill cuts of considerable height.

Another major refinement to the corridors was the routing to the west of the northern portion of the Denali route (Corridor 3). This routing was advocated on environmental grounds in an attempt to reduce potential impacts on the caribou subherd calving area near Butte Lake. A final refinement consisted of realignment of the portion of the Corridor on the south side of the river (Corridor 2) in the Stephan Lake-Fog Lakes area to reduce potential environmental impacts to furbearers and waterfowl .

The main routes within the corridors remaining after the initial screening are shown on Plate 29 and are briefly described below:

(i) Parks Highway to Devil Canyon

This route follows the existing portion of the Alaska Railroad between Gold Creek and the intersection of the railroad with the Parks Highway just south of Hurricane. This route passes through Chulitna Pass from the Parks Highway and then parallels the Indian River to Gold Creek. The existing river valley is sufficiently wide to accommodate a road. From Gold Creek to Devil Canyon, the route lies south of the Susitna River, paralleling the river on a high ridge.

(ii) Devil Canyon to Watana - South Side of Susitna River

This route generally parallels the Susitna River and traverses east-west from Devil Canyon to Watana. The topography is mountainous and the route involves the most difficult construction of the three routes, requiring a number of sidehill cuts in rock and soil. This route also includes the environmentally sensitive Stephan Lake and Fog Lake areas.

(iii) Devil Canyon to Watana - North Side of Susitna River

This route generally parallels the Susitna River and traverses east-west from Devil Canyon to Watana. This route is mountainous and includes terrain at the highest elevations of all routes, however construction of the road would not be as difficult as the route between the damsites south of the Susitna River.

(iv) Denali Highway to Watana

This route connects the Denali Highway with the Watana site and runs in a north-south direction. This route is the easiest to construct of the alternative routes. The terrain is relatively flat with a few wetlands involved. This route would not require any major bridges.

11.6 - Description of Basic Plans

From the three routes remaining after the initial screening, eight plans were developed. These plans were evaluated in more detail than originally planned in the original POS, as a result of information and assessments conducted during the study program, the concerns of state agencies, and recommendations of the Susitna Steering Committee (refer to Appendix D). The additional investigation and evaluations consisted mainly of environmental field work, geologic mapping and subsurface borings.

The plans are presented below and are also shown schematically in Figures 11.2 through 11.5.

(a) Plan 1

This plans utilizes a roadway from the Parks Highway to Watana Dam along the South side of the River. This access plan is based on materials such as cement and steel being brought into the State through the port of Whittier. Food and other camp supplies would be imported through Anchorage via container, and fuel directly from Kenai to Anchorage via existing pipeline. These materials and supplies would then be carried by rail to a railhead and storage area at Gold Creek. At Gold Creek, materials would be transferred to trucks for transport by road to the site. Other materials and supplies would be transported by truck from the Parks Highway. An alternative for fuel supply would be rail haul from the refinery at North Pole, Alaska.

(b) Plan 2 - All Rail

This plan would serve both damsites by a rail line. This alternative would essentially preclude public access. Construction planning for this mode of access would be based on trains being broken down and cars dropped on the siding at Gold Creek. An engine and train crew would be stationed at Gold Creek which would allow shuttle cars from Gold Creek to the project site on a daily basis. Passenger rail service would be required daily. If public access is desired after construction, the rails could be removed and the road bed graded into a single lane road with turnouts.

(c) Plan 3

This plan envisages the use of a combination of rail and road transportation. Construction activities at Watana would be served from a railhead and storage area at Cantwell by truck across the Denali Highway and along a newly constructed road from the Denali Highway. Construction at Devil Canyon would be served by road from a railhead at Gold Creek and road access from Gold Creek to the Parks Highway. This plan does not include a connection between the two dams.

(d) Plan 4

This plan serves Watana by truck from a railhead at Cantwell and Devil Canyon by rail from Gold Creek. In the plan, there is no connection between dams.

(e) Plan 5

This plan serves both dams by road from a railhead at Gold Creek. The route is located on the south side of the river to Devil Canyon with a major bridge downstream from the dam site, then follows the north side of the river to Watana. There is a road connection to the Parks Highway from Gold Creek.

(f) Plan 6

This plan is identical to Plan 4 except that a service road for maintenance purposes is included on the north side of the river between the two dams.

(g) Plan 7

This plan is the same as Plan 3 except that a service road would be provided along the north side of the river as in Plan 6.

(h) Plan 8

This plan is the same as Plan 5 except there is no road connection to the Parks Highway. A newly constructed road would service Devil Canyon from Gold Creek on the south side of the river. A major bridge would be required downstream of Devil Canyon and a new road on the north side of the river would connect the two dams. This alternative plan precludes public access.

11.7 - Additional Plans

Following selection and evaluation of the eight plans described above, presentations were made to the Alaska Power Authority and the Susitna Hydroelectric Project Steering Committee. These presentations and subsequent discussions resulted in the addition of the three plans described below.

(a) Plan 9

This plan is the same as Plan 8 except access between Gold Creek and Devil Canyon is by rail along a similar route, and the railhead is located at Devil Canyon instead of Gold Creek.

(b) Plan 10

This plan is identical to Plan 9 except that the road connecting Devil Canyon and Watana is on the south side of the Susitna River.

(c) Plan 11

This plan utilizes a railhead at Cantwell, road access via the existing Denali Highway, a road from the Denali Highway to Watana and a road from Watana to Devil Canyon on the north side of the River.

These plans are shown schematically in Figures 11.6 and 11.7.

Plans 9 and 10 suggested by the Steering Committee as a means to reduce accessibility to the area thus avoiding the introduction of adverse environmental impacts into the Susitna Basin.

Plan 11 was added as a possible way to provide access from only one area while also alleviating the socioeconomic impacts the west side communities would feel from any access road from the west.

11.8 - Evaluation Criteria

The specific objectives of the selected access plan are described in Section 11.2. The criteria used to assess the degree that any given plan satisfies these objectives are described in the following paragraphs.

(a) Construction Schedule

It is essential that the selected access plan be adequate to meet the overall project scheduling requirements. The load forecasts described in Section 5 together with the examination of the existing system and future generating options indicated a requirement for first power from Watana in 1993. A delay in the on-line date by one year would mean that another source of fossil fuel generation would have to be constructed, combined with retirement of some fossil fuel generation a year later, into the reserve category. In terms of present worth, a delay of one year would increase the cost of the project by approximately $50 million.

Analysis of the construction schedule requirements for Watana demonstrates that all-weather access route to the site is required by mid-1986 if the on-line date of 1993 is to be maintained. For the purposes of these studies, it has been assumed that an FERC licence to construct the project will be received at the beginning of 1985, and the start of permanent work on the project will coincide with this date. In order to meet all the mid-1986 requirements, it is obvious that an access route to the site would have to be constructed within approximately 18 months.

A preliminary evaluation of the construction period for completion of the access plans is presented below.

Plan	Origin for Watana Access	Approximate Construction Period (years)
1	Parks Highway	3-4
2	Gold Creek	3-4
3	Denali Highway	1
4	Denali Highway	1
5	Parks Highway	3-4
6	Denali Highway, Gold Creek	1
7	Denali Highway, Parks Highway	1
8	Gold Creek	3
9	Gold Creek	3
10	Gold Creek	3
11	Denali Highway	1

It is apparent from the above that only Plans 3, 4, 6, 7 and 11 could be constructed within the 18 month period required to maintain the overall project schedule. Since this would severely limit the selection process, a scheme was developed to provide initial access to the Watana Site within the framework of regulatory and scheduling restraints. This scheme, described in more detail in Appendix D involved construction of a pioneer road to Watana from either Gold Creek or the Parks Highway. The pioneer road would consist of a gravel based road with period passing turnouts and would be constructed on existing ground insofar as possible to avoid significant cuts or fills. Temporary floating Barley bridges would be used at river crossings, replaced by ice crossings in the winter. The analysis indicates that the pioneer road scheme will be sufficient to provide continuous access to the site within 18 months, and will be sufficient to support conctruction activities until the permanent access route is completed. Certain additional licensing and permiting requirements are associated with this scheme; these are discussed in Section 11.12.

The pioneer road scheme can be implemented with Plans 1, 2, 5, 8, 9 and 10, therefore all 11 plans can be considered equivalent in terms of their ability to meet initial project requirements.

(b) Construction and Logistics Costs

For the purposes of this evaluation, construction costs include the cost of constructing the access facilities, adjusted for any differences in cost of constructing the Susitna project itself which relate to the particular access plan under construction. Logistic costs are the costs associated with transporting, labor, fuel, equipment, materials and supplies to construct the two power developments.

(c) East of Operation and Maintenance

This criteria relates to the relative ease of operation and maintenance of the two developments after construction is complete. Initial planning envisages operation of both developments from Watana for several years after Devil Canyon is brought on-line, after which time, both projects will be operated remotely from a central location. Maintenance of two projects of this size and complexity will obviously be an important consideration. Duplication of maintenance facilities and staff at both sites would result in a substantial increase in the annual costs of the overall development. The most economic scheme, given the sequence of development, would be to establish an operation and maintenance facility at Watana, with a reliable means of access to Devil Canyon 32 miles downstream. In this regard, access plans with a road connection between the two sites have been evaluated as being superior in terms of ease of operation and maintenance than plans without a road connection.

(d) Flexibility in Construction Logistics and Transportation

This criterion is used to evaluate the extent to which an access plan contributes to the maintenance of a reliable and flexible logistic support system during construction of Watana and Devil Canyon.

For the Susitna project, a fundamental consideration is whether or not to provide a road connection to a major highway. For this evaluation, the following alternatives have been considered:

- a road connection either to the Parks Highway or Denali Highway;

- rail access only from Gold Creek;

Plans 1 through 10 described in Section 11.3 all include a railhead and storage area at Gold Creek. Accorindgly, plans incorporating a road connection with the Parks or Denali Highways obviously provide greater flexibility and reliability in case of a transportation description involving the Alaska Railroad, compared to plans with "rail only" access. Specific considerations are as follows:

- Any breakdown in the rail system would result in a loss of all ground transportation, in the absence of an alternative road system. The increased risk of delays has an associated cost penalty. An analysis has been undertaken to quantify the risks associated with rail access only. The methodology for this risk analysis is presented in Appendix D.

- The availability of two possible manes of transportation will undoubtedly be reflected in lower and more competitive bids for construction, supply and service contracts, since contractors would otherwise include some contingency to cover trnsportation disruptions. Although significant, this aspect is difficult to quantify.

(e) Environmental Considerations

Exclusive of socioeconomic considerations, the objective is to develop an access plan which minimizes adverse changes to the natural environment. The criteria used to assess the degree to which any plan meets this objectives are described below.

(i) Effects of Big Game

A primary concern associated with the selection of an access plan is the potential effect on the Nelchina caribou herd, specifically the subpopulation of approximately 1,000 animals that inhabit the northwestern section of the Upper Susitna Basin. The impacts of hunters on moose and bear are also considered but as secondary concerns. These impacts can be greatly lessened by selecting a route other than the access from the Denali Highway.

(ii) Effects of Fisheries

In the case of resident fisheries, there are relatively isolated lakes (Butte Lake, Big Lake) and streams in the northwestern section of the Upper Susitna Basin, and the Fog Lakes area that would receive additional angling pressure if road access was provided. These impacts can be lessened by avoiding access from the Denali Highway and the route on the south side of the Susitna River between the damsites.

Since Devil Canyon acts as a natural barrier to anadromous fish migration, there is no concern regarding the effect of improved access on this resource upstream of Devil Canyon. However, Indian River and the Susitna River below Portage Creek, are important for salmon. Any access plans that follow or cross these rivers could affect salmon directly through habitat disruption (i.e., sedimentation) or indirectly through increased fishing pressure. These impacts could be lessened by avoiding road access paralleling the Indian River.

(iii) Effects on Furbearers

Wetlands, important to furbearers, have been identified between the Parks Highway and Gold Creek, near Deadman Mountain, near Deadman and Big Lakes and the Upper Deadman Creek. In addition, the Fog Lake - Stephan Lakes wetlands complex is a valuable furbearer habitat. A red fox denning complex has also been identified south of Deadman Mountain. Any access road crossing through these areas has the potential for negative impacts on furbearers. Impacts on furbearers would be least by selecting access from Gold Creek to Devil Canyon on the south side of the Susitna River and on the north side of the River between the dam sites.

(iv) Effects on Birds

Heavily forested areas between the Parks Highway and Devil Canyon along riverbanks are productive avian habitat. Construction through these areas would disturb this habitat.

(v) Effects on Wilderness Setting

The Upper Susitna Basin is presently in a state of wilderness to semi-wilderness. Although continued intrusion with ATV's from Denali Highway, potential development of native lands and the establishment of the Indian River remote and disposal site have the potential of changing the character of sections of the basin, improved public access and construction of the Susitna Hydroelectric Project will produce a major alteration in the remoteness of the area. Natural resource agencies and the local public have expressed a desire to maintain the status quo to the maximum extent possible. People from the urban centers of Anchorage and Fairbanks have expressed a stronger desire to provide road access and open the area for recreation development. The status quo of the area would be retained to the greatest extent by providing only rail access to the damsites.

(vi) Effects on Archaeological Resources

Archaeological resources are likely present along all access routes. The route from Denali crosses a substantial area of high archaeological potential. The thin soil and lack of vegetation result in a high potential for impacts to resources along this route. The other access routes are believed to be less sensitive. Avoidance of the Denali access link lessens the probability of the disturbance of archaeological sites.

(f) Social Considerations

(i) Native Landowners

Native organizations have selected land surrounding the impoundment areas and south of the Susitna River between Devil Canyon and Watana damsites. To allow for increased opportunity to develop either lands on the south side of the river, the native landowners have expressed a strong desire to have a Susitna access road along the south side of the river from Watana to Gold Creek, ultimately connecting to the Parks Highway. It is considered that the basic native preferences would be met by providing road access to both damsites.

(ii) Local Community Preferences

Since the local communities are likely to receive many of the disbenefits with few of the benefits of a Susitna development, the objective to accommodate local community preferences has been included in the access plan selection process. The criteria used in assessing the degree to which this objective is met is divided into four areas due to the differences in community preferences (refer to Appendix D).

- Cantwell

The community of Cantwell desires economic stimulus and is in favor of the economic changes that could result from having a major construction project in the area. The desired stimulus could be achieved by providing road access to the Denali Highway with a railhead at Cantwell.

- North of Talkeetna

The communities along the railroad north of Talkeetna are opposed to development in the area and strongly prefer a maintenance of the status quo. These communities have expressed a desire for rail access only, although existing conditions in these communities would probably be disrupted least with a plan involving road access only from the Denali Highway.

This conclusion is based on the consideration that if rail access only is provided, the practicality of a self contained family-status community at either of the sites would be greatly diminished and a single-status only camp facility would likely be established. If this were to be the case, workers would tend to locate their families in the nearest communities, thus increasing the impacts in these communities.

- Talkeetna/Trapper Creek

Although attitudes are somewhat divided, the majority of residents of the communities of Talkeetna and Trapper Creek prefer a maintenance of the status quo. This can be most easily accomplished by providing access via Denali Highway.

- Willow/Wasila Area

The residents in this area are more in favor of economic development than in other areas.

- Indian River Land Disposal Sites

In 1981, a total of 75 remote state land parcels were awarded by lottery in the Indian River area. Of these, 35 were staked in the summer of 1981. The 35 land holders were contacted by letter through APA Public Participation Office. Of the 12 responses received to date, 11 favored retention of the remote status of the area and one favored road access to the area. This area would be most affected by road access from the Parks Highway and least affected by access from the Denali Highway.

(g) Agency Concerns

Correspondence, meetings and interaction with the various agencies involved with the Susitna Hydroelectric Project Steering Committee occurred throughout the study. Agency comments have been considered in the evaluation. The concerns of the agencies have generally related to environmental issues, with the emphasis on biological and land use impacts. Therefore, evaluation in terms of the environmental criteria discussed previously is considered to generally include agency concerns. It is considered that the resource agencies favor a rail only access plan with a major opposition towards road access from the Denali Highway.

(h) Transmission

Access plan selection has been coordinated with the transmission line studies. The transmission line studies to date have identified two corridors, one north of the Susitna River and one south of the Susitna River from Watana to Gold Creek. Although corridors run along the river, there is flexibility to expand the corridor to include the access road when the decision on which access route will be constructed is made. Due to more stringent engineering criteria of lines and grades for road alignments, it was decided that the selection of a transmission line route would occur subsequent to the access road selection.

The results of the transmission studies has also established that if the northern Denali access route is selected, the transmission line would not follow that route due to excessive cost and adverse visual impacts.

(i) Recreation

In meetings, discussions, and evaluation of recreation plans, it has become apparent that the various recreation plans are sufficiently flexible to accommodate any access route selected. No single route was identified which had superior recreational potential associated with it. Therefore, compatability with recreational aspects was essentially eliminated as an evaluation criteria.

11.9 - Evaluation of Access Plans

The 11 access plans evaluated on the basis of the criteria described in Section 11.8 have been grouped in accordance with the following categories in order to clarify the presentation.

Category	Plan Numbers
Plans providing access from both Parks and Denali Highways	3 and 7
Plans providing access from Parks Highway only	1 and 5
Plans providing access from Denali Highway	4, 6 and 11
Access from Gold Creek only	2, 8, 9 and 10

In addition to the specific considerations outlined in the following paragraphs, a major concern for all access plans is the creation of access to areas previously inaccessible or relatively inaccessible. Such access would lead to impacts to furbearers through increased trapping pressure and to big game through hunting pressure. In addition, detrimental effects could occur to all wildlife through disturbance and destruction of habitat by ATVs. Cultural resources would also be vulnerable to amateur collectors and ATV traffic.

(c) Access to Both Parks and Denali Highways (Plans 3 and 7)

(i) Cost

The costs of the 11 alternative access plans are summarized in Table 11.1. Given the preliminary nature of the field data used to develop construction costs, construction cost differences of less than $10,000,000 (approximately 5 to 10 percent of the cost of the alternatives examined) should not be considered significant.

Maintenance costs are a small portion of construction costs, and large variations in maintenance costs will have negligible influence on overall costs. The logistics costs are based on current freight rates and vary by less than 10 percent for all plans. The personnel shuttle costs and contingency risk costs are necessarily approximate but are adequate for comparison purposes. When comparing the total costs, the plans were considered equal if the total costs were within $20 million, and a definite cost advantage was considered if there was a $50 million difference.

On the basis of the foregoing, Plan 3 is comparable to the minimum cost alternative of any of the plans. Plan 7 has approximately a $60 million cost disadvantage compared to Plan 3.

(ii) Ease of Operation, Maintenance and Construction Flexibility

Access Plan 3 does not meet the ease of operation and maintenance criterion because it does not have a connecting road between Watana and Devil Canyon. Access Plan 7 does meet the ease of operation criteria by having a connection service road between the two sites.

Plans 3 and 7 both satisfy the criteria for flexibility for construction logistics and transportation by having a road access connecting to a major highway.

(iii) Biological

The primary biological concerns for these two plans relate to the effects the road would have on furbearers, big game, and cultural resources.

A roadway from the Parks Highway would cross productive furbearer wetlands habitat between the highway and Gold Creek. The Denali segment of both these plans also crosses aquatic furbearer habitat near Deadman Mountain, Deadman and Big Lakes, and Upper Deadman Creek. In addition, a red fox denning complex south of Deadman Mountain within one mile of the proposed road is likely to be affected.

The primary concern relative to big game for both these plans is the Denali segment, which would pass through an area that has frequently been used by either major portions or all of the Nelchina herd and includes the calving and summer ranges of the northwestern subgroups of the Nelchina caribou herd. The route also lies across the late summer migration route of caribou moving toward Butte Lake and Gold Creek and parallels a traditional spring migration route southward to the Susitna River.

The direct effects upon this group of caribou, should Plan 3 be implemented, include disturbance to cows and calves during the road construction period, a disturbance and possible impediment to caribou migration as a result of increased traffic in the area, and the possibility of direct mortality from road kills. However, the presence of the road should not interfere with migration, since caribou are known to cross roads. Moreover, interference with the calving areas could cause a major adverse impact on the females who show an affinity to traditional calving grounds.

Of greater importance than these factors, however, are the indirect consequences to this group of caribou of increased access to its range. An access road across this alpine tundra would provide the opportunity for all terrain vehicles to push a network of unplanned trails throughout the range of this subherd. This new access would cause disturbance and increased mortality to these caribou from their contact with vehicles, campers, and hunters. Thus, there is a chance that this route could lead to partial abandonment of important caribou habitat.

The actual magnitude of impact is difficult to assess since it depends on the somewhat unpredictable behavior of both caribou and man. With an increased emphasis on management of the area and stringent hunter control, it is technically possible to lessen the potential extent of impact. It is expected, however, that resource agencies would be apprehensive about the success of any mitigation plans and would strongly resist any road access from the Denali Highway.

(iv) Social Considerations

Without mitigating measures, access plans with a roadway originating from the Parks Highway could significantly impact the westside communities in terms of demand for increased services, changes in population, housing availability, government expenditures and revenues, labor demand, and unemployment. There will also be significant effects on construction, retail trade, and tourism.

Many of these changes will occur as construction workers attempt to relocate to the communities near the construction site. Depending upon commuting modes to the camp, there could be a large increase in vehicular traffic in the area.

These access plans also include a road from the Denali Highway. As such, many of the impacts which would be felt in the west side communities of Talkeetna, Trapper Creek, and Sherman would also occur in Cantwell. With a road from the north, it is expected many of the workers would settle in Fairbanks, thereby reducing some of the impacts which the west side communities would experience.

These plans meet the preference of the public in Cantwell as some changes will occur but will not meet the preferences expressed by those in the west side communities who desire no change.

However, road access connecting the Denali and Parks Highway would create extensive public access following construction thus creating the maximum change in the status quo of the area.

As discussed under Section 11.13, it is considered that mitigation measures can be implemented to lessen the effects on the west side communities of Talkeetna and Trapper Creek. With road access from the Parks Highway, change in the remoteness of Gold Creek and the Indian River Land Disposal sites will occur regardless of mitigation.

(b) Access from Parks Highway Only (Plans 1 and 5)

(i) Costs

Access Plans 1 and 5 are both comparable to the minimum cost alternative (Table 11.1).

(ii) Ease of Operation and Construction Flexibility

Both Access Plans 1 and 5 satisfy the ease of operation criteria by having a road directly connecting both sites. Both Access Plans 1 and 5 satisfy the flexibility criteria by having a road connection with a major highway.

Access Plans 1 and 5 involve a shorter haul distance compared to any alternative having access via Denali Highway.

Anchorage has been identified as the most viable port of entry for the majority of the materials and supplies (__). When comparing Access Plans 1 and 5, with plans having access from the Denali Highway, logistics and cost advantage over any access from the Denali Highway. With the majority of materials and supplies coming from Anchorage, the access route from the Denali Highway would involve an additional haul of approximately 52 miles to Watana when compared to an access from the Parks Highway. The additional 52 miles of haul to Watana, for a Denali access alternative, would be a disadvantage in long-term operation and maintenance.

(iii) Biological Considerations

The primary concerns with access from only the Parks Highway were discussed in (a) above. Briefly, the concerns are the potential impact to furbearer habitat between the highway and Gold Creek and potential degradation of fisheries habitat in the Indian and Susitna

Rivers. Of lesser concern is the disturbance of moose and bear populations and removal of their habitat caused by the northside connecting road in Plan 5.

In addition to these, Plan 1 includes a connection on the south side of the Susitna River between the two damsites. This road would pass near and through extensive wetland areas in the Stephan Lake-Fog Lake area. These wetlands provide habitat for furbearers and waterfowl and support a large, year-round concentration of moose. Because this area is currently relatively inaccessible, potential impacts include removal of habitat and increased mortality through hunting and trapping.

(iv) Social Considerations

Evaluation of these plans from a socioeconomic aspect reveals that Plans 1 and 5 will result in the greatest impact to the west side communities. Because access is provided from the west only, the majority of the impacts would be felt in the west side communities. There would be a greater tendency for people to relocate in the communities and perhaps in Anchorage and a lesser tendency to live in the Fairbanks area. There would be some impacts to the Cantwell area, but fewer than with a road from Denali. Impacts would be the same as discussed in (a) above.

In terms of public preference, these plans least meet the desires of people living in the project area. The plans would cause the greatest change in the Talkeetna-Trapper Creek area (where residents have expressed negative attitudes toward social change) and would minimize impacts to the Cantwell area (where residents have expressed a desire for change). The Indian River land disposal site and Gold Creek would experience the greatest change with the selection of this plan.

(c) Access for Denali Highway (Plans 4, 6 and 11)

(i) Costs

Table 11.1 indicates that Plan 4 is comparable to the least cost alternative (Plan 5). The cost of Plan 6 is approximately $40 million greater than that of Plan 4 and the cost of Plan 11 is approximately $35 million greater than that of Plan 4.

(ii) Ease of Operation and Construction Flexibility

Plan 4 does not satisfy the ease of operation criterion due to the absence of a road directly connecting the two dam sites. Plans 6 and 11 both have a road directly connecting the dam sites, therefore both plans satisfy the ease of operation criterion.

Plan 4 only partially meets the construction flexibility criterion. Plan 4 includes a road connection to a major highway for the Watana development but not for the Devil Canyon development. Access Plans 6 and 11 both satisfy the flexibility criteria by having a connection to a major highway.

(iii) Biological Considerations

These three plans all involve road access from Denali Highway to Watana damsite. The potential biological and cultural impa asso- ciated with this route were discussed under (a) above. Basi ılly, impacts could occur to portions of the Nelchina caribou herd through increased hunting mortality and potential interference with migration and calving. Increased access and trapping pressure could also impact furbearers. In addition, because of treeless topography and shallow soil, disturbance and removal of any cultural resources could result.

Plans 4 and 6 involve construction of a rail connecting from Gold Creek to Devil Canyon. No major environmental problems were identified along this portion of the route. The connection road on the north side of the Susitna River between the two dams was discussed under (b) above, the only environmental concern being the crossing of moose habitat.

(iv) Social Considerations

Plans 4, 6 and 11 involve the major access point of origin on the Denali Highway, rather than the Railbelt Corridor. Workers' families would tend to locate more communities, including Cantwell and Fairbanks. Due to the rail access from Gold Creek, there would still be changes in the west side communities, but fewer than with a road originating from the Parks Highway. Plan 11, involving access from Denali Highway only, would cause the greatest number of changes in the Cantwell and Fairbanks area and fewer changes to the west side communities. These changes would be the same as described in (a) above.

(d) Access from Gold Creek Only (Plans 2, 8, 9 and 10)

Table 11.1 indicates that the total cost of Plan 8 and 9 are respectively $15 and $30 million greater than the least cost alternative, Plan 5. The substantial savings in construction costs are offset by increased personnel shuttle costs and an allowance for contingency risk. The cost comparison also shows that the total costs of Plans 2 and 10 are $55 million and $40 million more than that of the least cost alternative.

(i) Ease of Operation and Construction Flexibility

Access Plan 2 meets the criterion for ease of operation since the dams are directly connected with a rail route. Access Plans 8, 9 and 10 partially satisfy the ease of operation and maintenance criteria. These plans have a road directly connecting the two dam sites, however, they do not have a connection to a major highway.

This reduces the flexibility in operation and maintenance of the sites as discussed in Section 11.8.

Access Plans 2, 8, 9 and 10 do not satisfy the flexibility criteria for construction as they do not have a road connection to a major highway.

(ii) Biological Considerations

These plans all preclude access from the Parks Highway or Denali Highway; therefore, the impacts associated with increased access are substantially reduced.

Plans 2 and 10, which involve connections between Watana and Devil Canyon on the south side of the Susitna River, have as the major potential environmental impacts, the disturbance of wetland areas near Stephan and Fog Lakes, as discussed under (b) above.

The overall reduction in access and the fact there is no access connecting with the Denali Highway to the north indicates these plans would result in the least number of impacts to biological and cultural resources.

(iv) Social Considerations

These plans all involve access from the west only, the only difference being road or rail, and if rail, the distance into the basin the railroad extends. As such, impacts would again be concentrated on the west side communities. These impacts would likely be concentrated in the Gold Creek area as well as Talkeetna and Hurricane because of their location at rail-highway intersections. The Cantwell and Fairbanks areas would be less affected as there would be no northerly access.

The public has expressed a preference for a rail access and a maintenance of the status quo. Although rail access would best maintain the status quo of the Upper Susitna Basin in general with the rail access, significant changes could occur in the Talkeetna/Trapper Creek area as discussed in Section 5.1 (e).

These plans would not meet the public preferences expressed by Cantwell residents.

11.10 - Identification of Conflicts

From the evaluation presented in Section 11.8, it is apparent no single plan meets all the objectives or satisfies all the criteria established as part of the study. The basic conflicts identified were:

(a) Social and Biological Considerations vs Construction and Operation Logistics

Rail or road access from a railhead at Gold Creek without road access from a major highway would limit social and biological changes in the immediate project area and retain the status quo to the greatest extent possible. This option is in direct conflict with the requirement to provide flexibility in construction logistics and transportation and to provide ease of operation and maintenance. The selection of such an option would increase the risk of high costs, schedule delays, and safety problems and decrease project reliability.

(b) Social vs Biological Considerations

Social and biological objectives are not in basic conflict since limited access to the project area is most desirable in both cases. If, however, the assumption is made that road access to a major highway will be provided, then a conflict arises. From the social/local public preference perspective, access from the Denali Highway is preferred. This plan would create the economic stimulus desired in Cantwell, reduce the potential for change in the Trapper Creek/Talkeetna area, while retaining the remoteness of the Indian River land disposal site and the railroad communities north of Talkeetna. The Denali access, however, is in conflict with biological objectives since it would allow access by hunters and ATVs to a large portion of the Upper Susitna Basin and create potential impacts on the Nelchina Caribou, other big game species including moose and bear, the fisheries in isolated lakes and streams and furbearer habitat. In addition, the potential for disturbance of archaeological sites in this area is greatest. Although mitigaion measures can be employed to reduce these potential biological impacts, it is considered likely that government resource agencies would be apprehensive about the success of any control programs and would thus be opposed to any access from the Denali Highway.

The selection of a Denali access plan could result in unacceptable delays in license approval or a subsequent rejection of this plan requiring a reassessment of access plans from the west.

Table 11.2 broadly summarizes the conflicts in the evaluation.

11.11 - Comparison of Access Plans

(a) Access from Railhead at Gold Creek (Plans 2, 8, 9 and 10) vs Access from Major Highway (Plans 1, 3, 4, 5, 6, 7, 11)

Considerable cost, schedule, safety and reliability risks are associated with construction of a major project without road access to a major highway. On the other hand, road access to a major highway will create additional change in the status quo of the Upper Susitna Basin. If the decision is made to develop a large scale hydroelectric facility in the Upper Susitna Basin, it is considered essential that the orderly development and maintenance of the facility should be afforded a higher priority than maintenance of the status quo. Thus, access plans originating at a railhead at Gold Creek only are not recommended.

These considerations led to the rejection of plans not providing road access to a major highway.

Plans eliminated in this comparison: 2, 8, 9, 10
Plans remaining: 1, 3, 4, 5, 6, 7, 11

(b) Access From Both Parks Highway and Denali Highway (Plans 3, 7) Vs Access from Only One Highway (Plans 1, 2, 4, 5, 6, 7, 8, 10, 11)

The plans which optimize transportation flexibility and ease of operation involve the initial construction of a road from Denali Highway to Watana damsite. To allow for improved logistics during the peak construction at Watana and throughout the construction of Devil Canyon, road access would also be created to the Parks Highway. The disadvantages of these plans are that they would create the maximum change in the status quo producing both the biological impacts associated with the Denali link and the social impacts associated with the Parks Highway link. These impacts are further intensified with both roads since the connection of the Parks and the Denali Highway would encourage hunters and tourists to drive the complete loop.

These plans are also more costly than the minimum cost alternatives. It is considered that the social and biological impacts that would result from these plans cannot be justified by the added transportation flexibility and ease of operation benefits associated with road access to both the Parks and Denali Highways.

These conclusions resulted in the rejection of the plans providing road access to both the Parks and Denali Highway.

Plans eliminated in this comparison: 3, 7
Plans remaining: 1, 4, 5, 6, 11

(c) Roadway Connecting the Dam Sites Directly (Plans 1, 2, 5, 6, 7, 8, 9, 10 11) vs No Roadway Connecting the Dam Sites Directly (3, 4)

Plans incorporating a road connecting the dam sites directly are clearly superior in terms of ease of operation and maintenance to plans which do not directly connect the dam sites. The access plans which do not connect the damsites directly do not have advantagesin any of the other, or combined criteria to warrant not eliminating these alternatives from further consideration.

These conclusions resulted in the rejection of plans not connecting the damsites directly.

Plans eliminated in this comparison: 3, 4
Plans remaining: 1, 5, 6, 11

(d) Access to Denali Highway (Plans 3, 4, 6, 7, 11) vs Access to Parks Highway (Plans 1, 5)

The main concerns associated with the Denali access are the potential effects on the Nelchina caribou herd, increased access to a large area of alpine tundra with the associated effects of disturbance by ATVs and disturbance of potential cultural resources.

Although there are some fisheries and furbearer concerns in the Indian River area associated with a Parks Highway access, from the biological perspective, Parks Highway access is preferred to a Denali Highway access.

From a social perspective, the Denali route is clearly superior to the Parks Highway route. The Denali route would promote the economic stimulus desired in Cantwell while reducing the influence on the community of Trapper Creek and Talkeetna which has expressed a desire to maintain the status quo. It is considered, however, that even with a Parks Highway access, mitigation in the form of self-contained construction camp facilities, regulation of commuter schedules and control of transportation modes can reduce or avoid many of the potential changes in Talkeetna and Trapper Creek.

With any access plan from the west, a major railhead would be located at Gold Creek creating sufficient local changes. With road access from the Parks Highway to Gold Creek, changes will also occur at Indian River and disposal sites.

Based on the above discussion, it is concluded that the Parks Highway access is preferable to the Denali access plan. This conclusion is based on the assumption that:

- if a Denali route were selected, it would be Plan 6 which would still result in significant social changes in the Gold Creek area;
- government resource agencies will be opposed to the Denali route with a likely 1-to-2 year delay in schedule or denial of permit resulting;
- changes in local communities can, to a large degree, be mitigated through controls imposed on contractor and construction workers; and
- controls would be very difficult to impose upon hunters and ATV operators who would utilize the Denali's route after construction.

The foregoing considerations resulted in the elimination of plans involving access from the Denali Highway.

Plans eliminated in this comparison: 3, 4, 6, 7, 11
Plans remaining: 1, 5

(e) Comparison of Plan 1 vs Plan 5

Plans 1 and 5 both commence on the Parks Highway near Hurricane and proceed through Chulitna Pass and along the Indian River to Gold Creek. From Gold Creek, both Plans proceed east on the south side of the Susitna River to the Devil Canyon site. At Devil Canyon, Plan 1 proceeds east on the south side of the Susitna River to the Watana site. Plan 5 crosses the Susitna River at Devil Canyon and proceeds east on the north side of the Susitna River to the Watana site. Access Plan 1 has potential for greater environmental impacts than Plan 5. Access Plan 5 has a slight cost advantage over Plan 1, also Plan 5 is slightly easier to construct due to the difficult terrain in the segment between Devil Canyon and Watana south of the Susitna River. The only advantage Plan 1 has over Plan 5 is in Native landowner preference. It is, therefore, concluded that the environmental cost and construction considerations outweigh the Native landowner preference, and therefore, Plan 1 is eliminated from further consideration.

11.12 - Recommended Access Plan

Based on the above discussion, it is recommended that:

- The selected access plan for the construction and operation of the Susitna Hydroelectric Project should comprise a road commencing near MP 156 on the Parks Highway, proceeding southeast crossing the Susitna River at Gold Creek, turning northeast to Devil Canmyon Dam site along the southern side of the Susitna River, crossing the Susitna River at Devil Canyon, an proceeding along the north side of the Susitna River to Watana Dam site (see Plate 30).

- To allow for continued access for project construction by mid-1987, a pioneer road (limited access) between Gold Creek and Watana Dam site be constructed commencing in mid-1983. The application for permits to construct this pioneer road be submitted to the State of Alaska and the Bureau of Land Management by August 1982, independent of the FERC license application.

- To mitigate against agency concerns in regard to the pioneer road concept and to avoid the possibility of public access to the project area in the event that the project is not built, road access between the Parks Highway and Gold Creek not commence until after FERC license approval. If the project does not proceed after the pioneer road is constructed, the road as such should be rendered impassable to future vehicular traffic.

- To minimize potential impacts to furbearers and fisheries resources in the Indian River and Susitna River areas, special construction techniques be utilized (including adequate bank stabilization, revegetation and restoration) when crossing wetland areas or when constructing in proximity to the Indian or Susitna Rivers.

- To minimize the effects of public access during the operation phase of the project consideration be given to controlling public access across Devil Canyon Dam. If access is provided east of Devil Canyon Dam site, restrictions should be placed on the use of ATVs and hunting.

- To assist in minimizing changes in the local communities of Talkeetna, Trapper Creek, Sherman and Curry, it is strongly recommended that subsequent decision on construction camp facilities, commuter modes, work incentives, and general policies incorporate a special effort to minimize the effects of construction on these local communities. Specific mitigation recommendations are included in Section 11.13.

The foregoing is based on the following assumptions:

- The pioneer road concept will be approved by government regulatory agencies since the pioneer road would not connect to any existing road before the issuing of a FERC license, thus not making the prior commitment to allowing public access to the Upper Susitna Basin.

- Although the native landowners have expressed a strong preference for road access from Parks Highway to both damsites along the south side of the Susitna River, their basic desires would be met by providing road access, from any direction, to their existing land holdings.

- Public access will be prohibited during the construction phase of the project. Also, the selection of Plan 5 offers some flexibility in regard to the degree and type of public access subsequent to 1993.

- Biological and social impacts will be mitigated through adoption of the recommendations presented in Section 11.13.

If permits to commence construction of the pioneer road are not obtained by mid-1983, it may be necessary to accept a 12- to 18-month delay in the on-line schedule or possibly revert to one of the less acceptable access plans which do not require a pioneer road.

11.13 - Mitigation Recommendations

The plan recommended by Acres does not satisfy all the evaluation criteria outlined in Section 11.2. In order to reduce potential impacts to biological and cultural resources and to alleviate socioeconomic impacts to the communities of Talkeetna, Trapper Creek, Sherman and Curry, the following mitigation measures are recommended:

- Permit only on-duty construction workers to have access to both the pioneer road and access road.

- After construction of the power development is complete, maintain a controlled access route beyond the Devil Canyon Dam. It is anticipated a cooperative agreement could be reached with BLM and ADF&G concerning the number of people permitted access to the areas and the cost of any control measures.

- The construction camp should be as self-contained as possible, thus limiting the number of workers who could otherwise bring their families to a nearby community and commute daily.

- Provide incentives to encouraged workers to work the longest time possible between leaves to minimize commuter traffic. Although the final schedule will not be known until labor agreements are established and construction commences, longer work periods between breaks can be advocated.

- Provide planning assistance if requestd to the communities of Talkeetna, Trapper Creek, Sherman and Curry to aid them in preparing for the effects of increased populations.

- Evaluate various commuter management policies and select the one which reduces impacts to the local communities. Socioeconomic impact assessment studies currently under way for the Susitna Project will provide important input data for evaluating possible commuter management policies.

- Utilize excavated cuts and other construction techniques to prohibit utilization of the pioneer road after construction of the access road. Areas used for the pioneer road which do not follow final road alignment should be reclaimed.

11.14 - Tradeoffs Made in the Selection Process

(a) Basis of Selection Process

From the natural resource and local public preference perspective, maintenance of the status quo is probably most favored. However, it is unrealistic to consider that a project the size of Susitna can be implemented without changing the existing character of sections of the Upper Susitna Basin.

Access to the dam sites is a complex and controversial issue. As such, it has received considerable attention from the study team, APA, resource agencies and the public. Although the studies have determined that there is no single access plan that satisfies all the project objectives and evaluation criteria, it has been possible to develop an access plan which provides a reasonable tradeoff of preference. These tradeoffs are essentially based on the following compromises:

- All entities must present a degree of flexibility, otherwise a satisfactory compromise is impossible.

- Whenever a specific objective is partially compromised, considerable effort is made during subsequent decisions to compensate.

- Any compromises made are clearly outlined such that decision makers reviewing the final recommendation are aware of negotiations to date.

(b) Tradeoffs Made in the Selection Process

(i) Engineering

Concessions made include:

- no road access from Denali Highway which would include a complete loop connecting Parks Highway with Denali Highway;

- no pioneer road to Parks Highway prior to the issuance of a FERC license;

- commitment to be prepared to make the pioneer road impassible if FERC license not granted;

- restrictions to be placed on worker commuting schedules and mode; worker incentives to be provided to minimize effects on local communities;

Objectives retained include:

- road access to both dam sites to allow for ease of construction, operation and maintenance of the project;

- maintenance of schedule through retention of the basics of the pioneer road concept.

(ii) Biological

Concessions made include:

- road access from Parks Highway affecting Indian River area and providing partial public access to the upper basin:

Objectives retained include:

- no access from Denali Highway which was considered to have the greatest potential for environmental impact;

- no route on the south side of the Susitna River between the dam sites, thus avoiding the sensitive Stephan Lake and Fog Lakes area;

- emphasis on construction mitigation when developing road link between Parks Highway and Gold Creek;

- retention of a degree of control on future public access by accepting the Parks Highway plan where, due to the terrain, private vehicles are basically restricted to the access corridor between Parks Highway and the Devil Canyon dam site. The degree and type of access east of Devil Canyon can be somewhat controlled by regulation of access across the Devil Canyon dam.

The alternative of not connecting to a major highway was considered to have the least net adverse biological impact. The ease of operation and maintenance and the construction flexibility criteria, as explained previously, was considered to outweigh this advantage. The mitigation measures and road management will reduce the adverse biological impacts associated with an access connection to a major highway, to a minimum.

(iii) Social

Concessions made include:

- road access to the Upper Susitna Basin;

- road access from Parks Highway which creates greatest potential for change in the Indian River land disposal site.

Objectives retained include:

- Through the implementation of a relatively self-contained construction camp, restriction of private vehicles from the construction site, implementation of mass transit modes for commuting workers, incentives to encourage workers to remain on site and controlled public access east of Devil Canyon following construction, it is considered that changes in the local communities of Trapper Creek/Talkeetna area will be minimized;

- Although the western communities favored a rail access, they also favored maintaining the status quo. It is our opinion that the recommended plan with its associated mitigation should produce less change in the Talkeetna/Trapper Creek area than an all-rail access plan.

  Overall consensus of the local community preference favored access from the Denali Highway. The advantages of the Parks Highway access over the Denali access in reducing the biological impacts is considered to outweigh the local community preference. In addition to the lessened biological impacts, the recommended plan better meets the preferences of Native landowners.

  The recommended plan does not fully meet the preferences of the Native landowners. They would prefer the access road between Devil Canyon and Watana be located on the south side of the Susitna River. The advantages of the road being located on the north side of the Susitna River, include, reduced biological impacts, the actual construction of the road is easier than if located on the south side, and the construction cost of the road is less. These advantages are considered to outweigh the Native landowner preference of having the road located on the south side of the Susitna River.

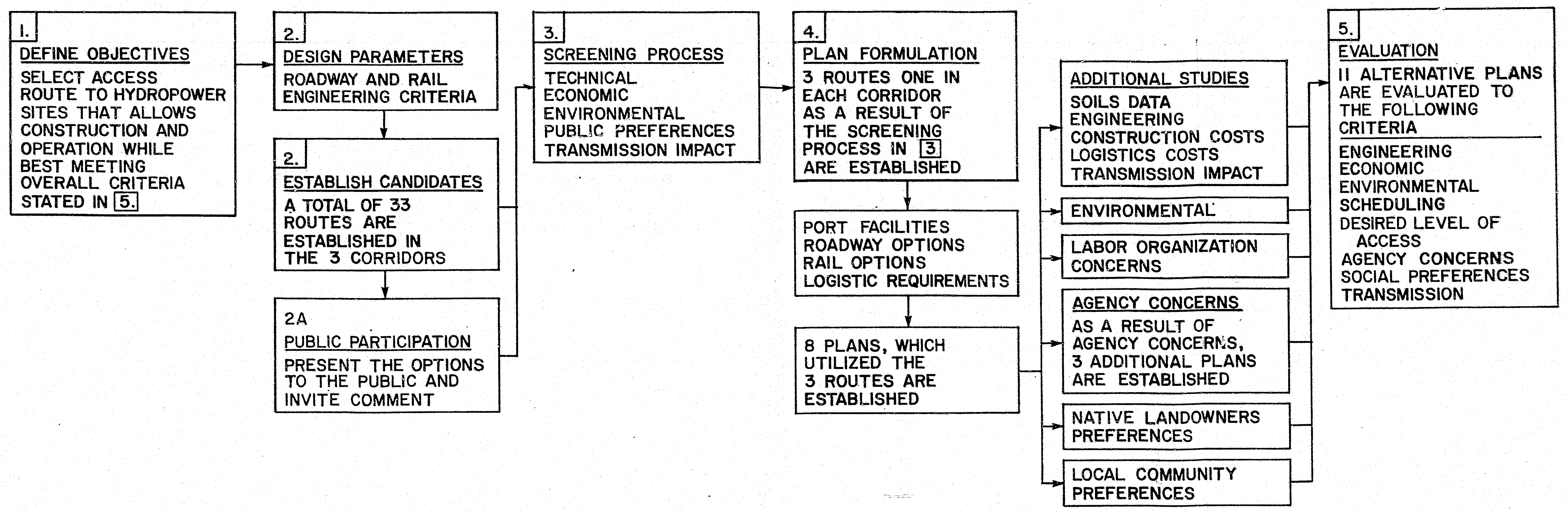
TABLE 11.1: SUSITNA ACCESS PLANS

PLAN	1	2	3	4	5	6	7	8	9	10	11
DESCRIPTION:	ROADWAY: PARKS HIGHWAY TO DEVIL CANYON & WATANA ON SOUTH SIDE OF SUSITNA	RAIL: GOLD CREEK TO DEVIL CANYON & WATANA ON SOUTH SIDE OF SUSITNA	ROADWAY: DENALI HIGHWAY TO WATANA, PARKS HIGHWAY TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA. NO CONNECTING ROAD	ROADWAY: DENALI HIGHWAY TO WATANA, RAIL, GOLD CREEK TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA. NO CONNECTING ROAD	ROADWAY: PARKS HIGHWAY TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA, DEVIL CANYON TO WATANA ON NORTH SIDE OF SUSITNA.	ROADWAY: DENALI HIGHWAY TO WATANA, RAIL, GOLD CREEK TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA. CONNECTING ROAD ON NORTH SIDE OF SUSITNA.	ROADWAY: DENALI HIGHWAY TO WATANA, PARKS HIGHWAY TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA. CONNECTING ROAD ON NORTH SIDE OF SUSITNA.	ROADWAY: GOLD CREEK TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA, DEVIL CANYON TO WATANA ON NORTH SIDE OF SUSITNA.	RAIL: GOLD CREEK TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA, ROADWAY DEVIL CANYON TO WATANA ON NORTH SIDE OF SUSITNA.	RAIL: GOLD CREEK TO DEVIL CANYON ON SOUTH SIDE OF SUSITNA. ROADWAY DEVIL CANYON TO WATANA ON SOUTH SIDE OF SUSITNA.	ROADWAY: DENALI HIGHWAY TO WATANA CONNECTING ROAD BETWEEN WATANA AND DEVIL CANYON ON NORTH SIDE OF SUSITNA.
MILEAGE OF NEW ROAD	62	58	70	60	68	102	111	54	58	53	86
CONSTRUCTION COST ($ x 1,000,000)	158	140	151	119	143	179	209	93	108	123	145
MAINTENANCE COST ($ x 1,000,000)	5	4	6	5	8	8	9	7	5	5	11
LOGISTICS COST ($ x 1,000,000)	215	210	231	230	214	230	231	214	216	214	258
SUBTOTAL ($ x 1,000,000)	378	354	388	354	365	417	449	314	329	342	414
PERSONNEL SHUTTLE COST ($ x 1,000,000)	0	25	0	10	0	0	0	25	25	25	0
CONTINGENCY RISK ($ x 1,000,000)	0	40	0	15	0	0	0	40	40	40	0
TOTAL COSTS ($ x 1,000,000)	378	419	388	379	365	417	449	379	394	407	414
CONSTRUCTION SCHEDULE	3-4	3-4	1	1	3-4	1	1	3	3	3	1
MAJOR BRIDGES	2	2	0/1	0	2	0	0/1	1	1	1	0

TABLE 11.2: IDENTIFICATION OF CONFLICTS

Criteria	1	2	3	4	5	6	7	8	9	10	11
Costs											
Minimize Costs	3	3	3	3	3	1	1	3	3	3	1
Ease of Operation and Construction Flexibility											
Ease of Operation and Maintenance	3	2	1	1	3	3	3	2	2	2	3
Construction Flexibility	3	1	3	2	3	2	3	1	1	1	3
Biological											
Minimize Biological Impacts	2	3	1	1	2	1	1	3	3	3	1
Social											
Accommodate Preference of Native Landowners	3	1	2	2	2	2	2	1	1	1	1
Accommodate Local Community Preference	1	2	2	3	1	3	2	2	2	2	3

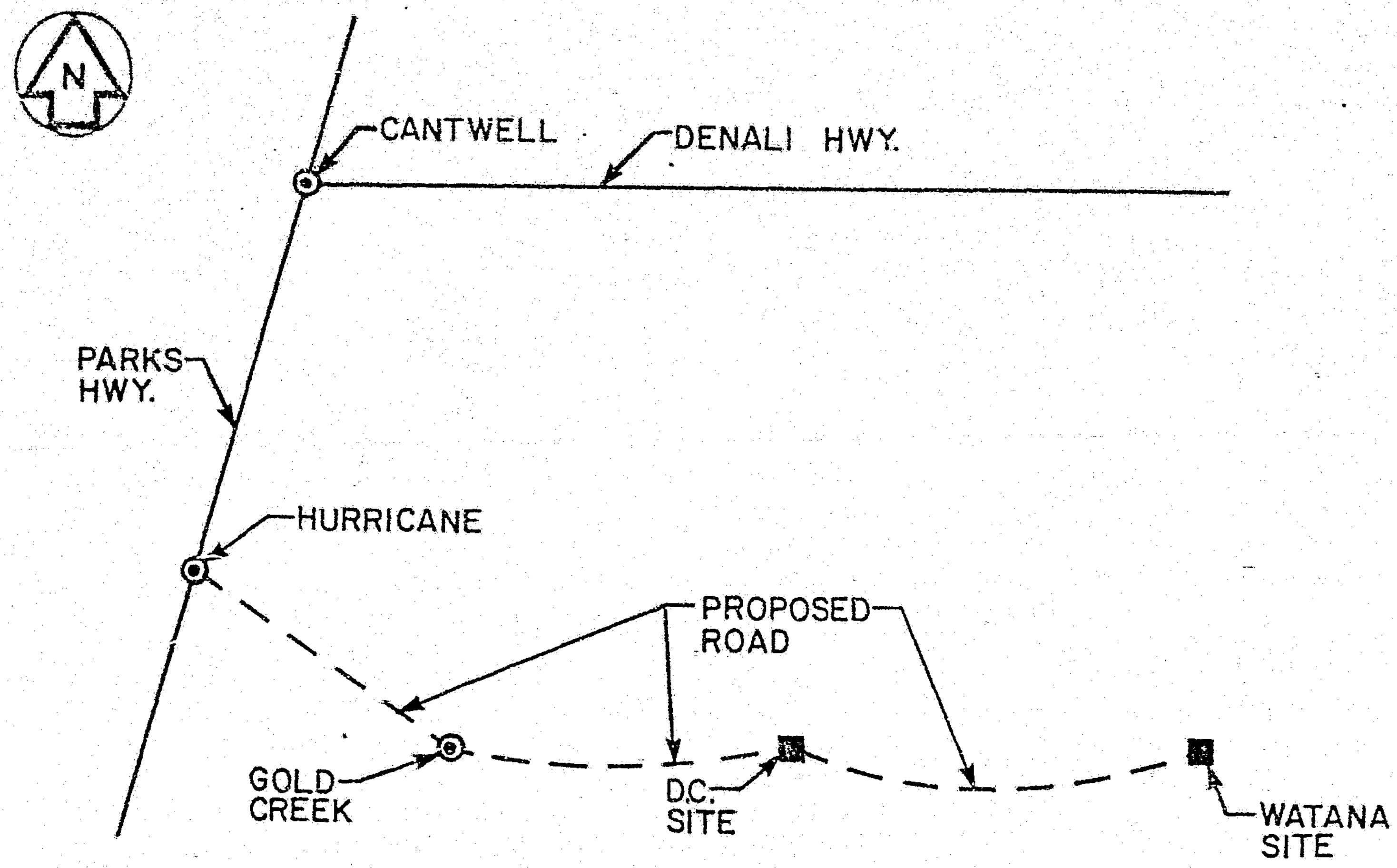
1 - Does not satisfy criteria
2 - Intermediate
3 - Satisfies criteria



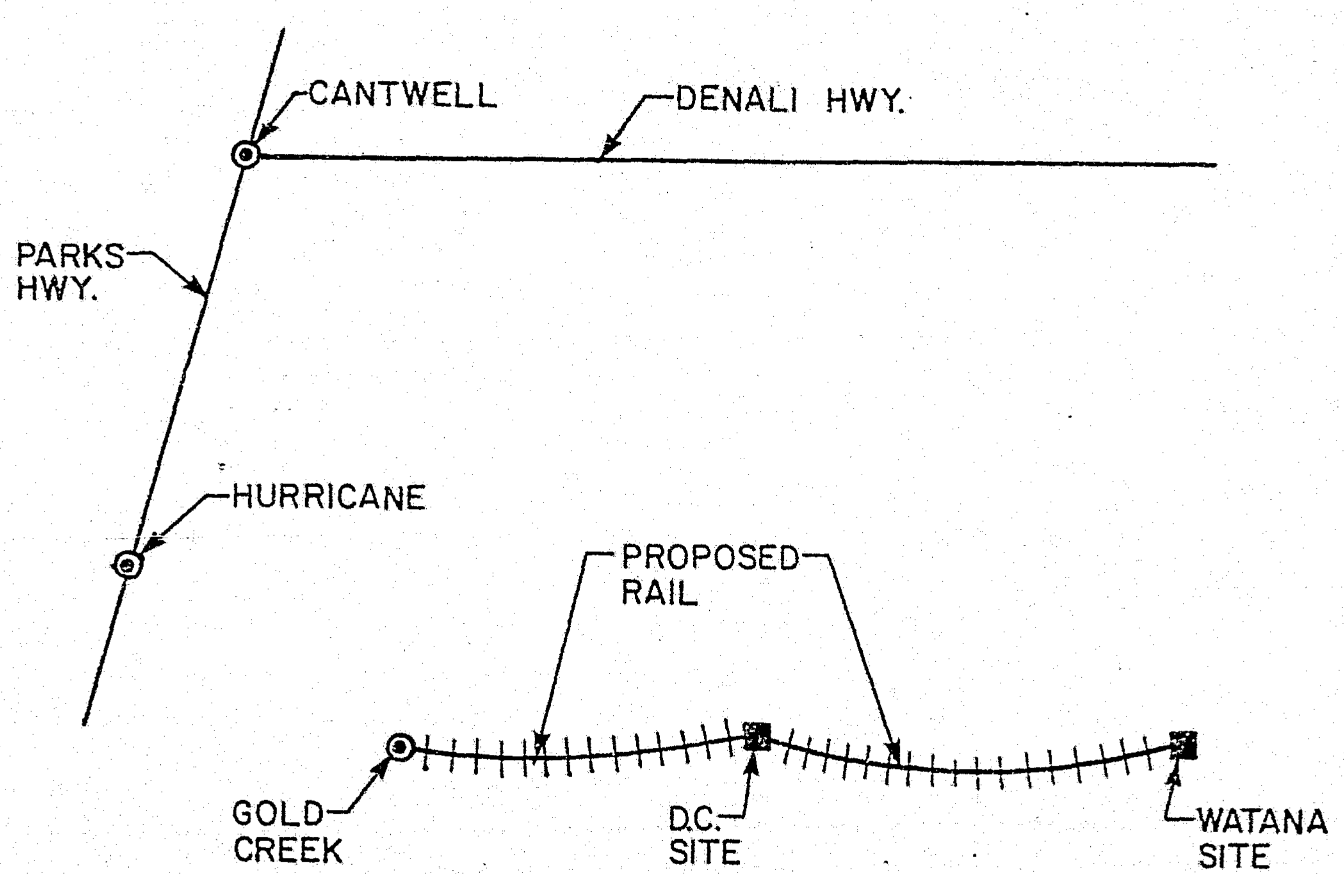
ACCESS PLAN SELECTION METHODOLOGY

FIGURE III.1

ACRES



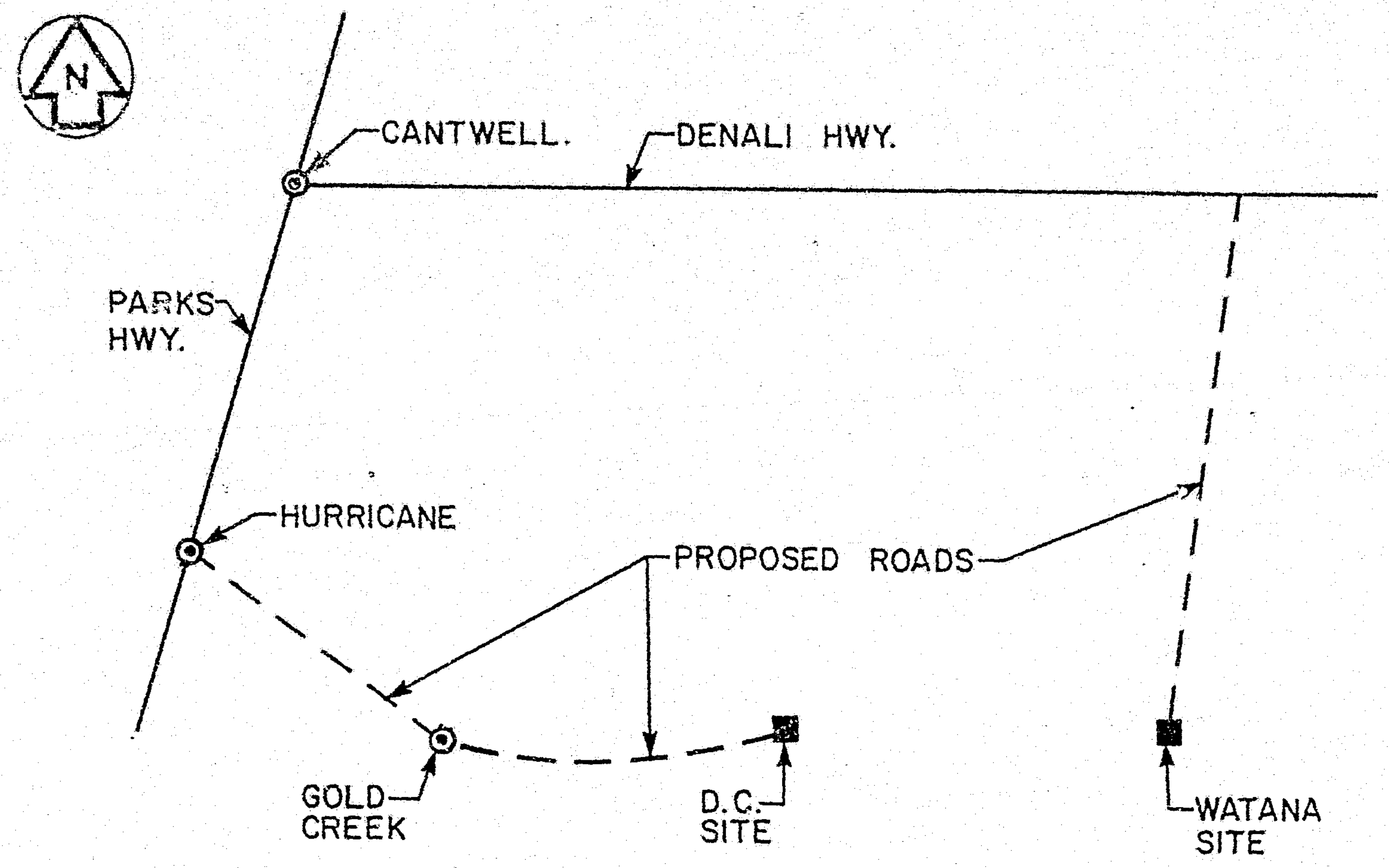
PLAN 1



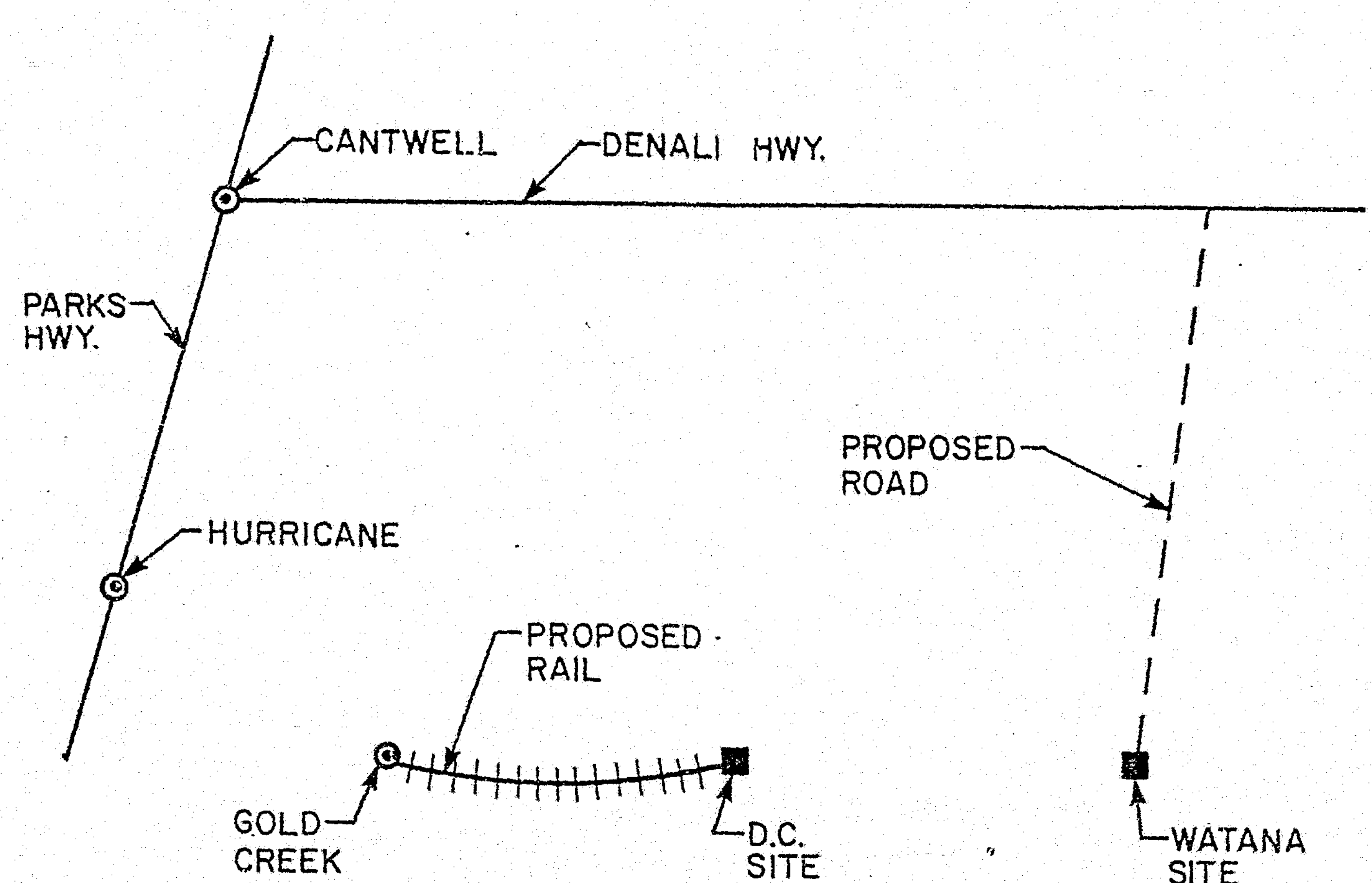
PLAN 2

FIGURE 11.2

ACRES



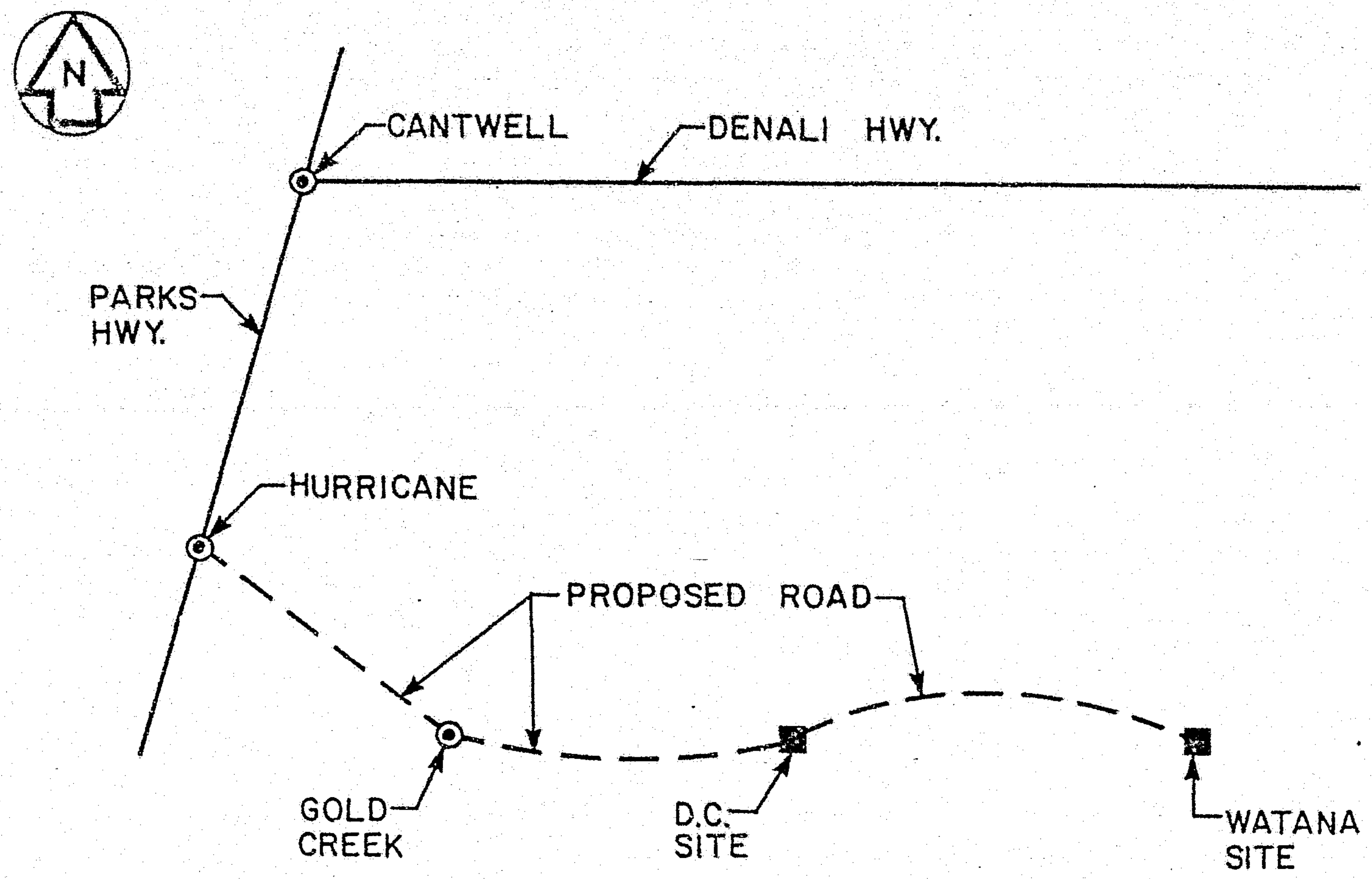
PLAN 3



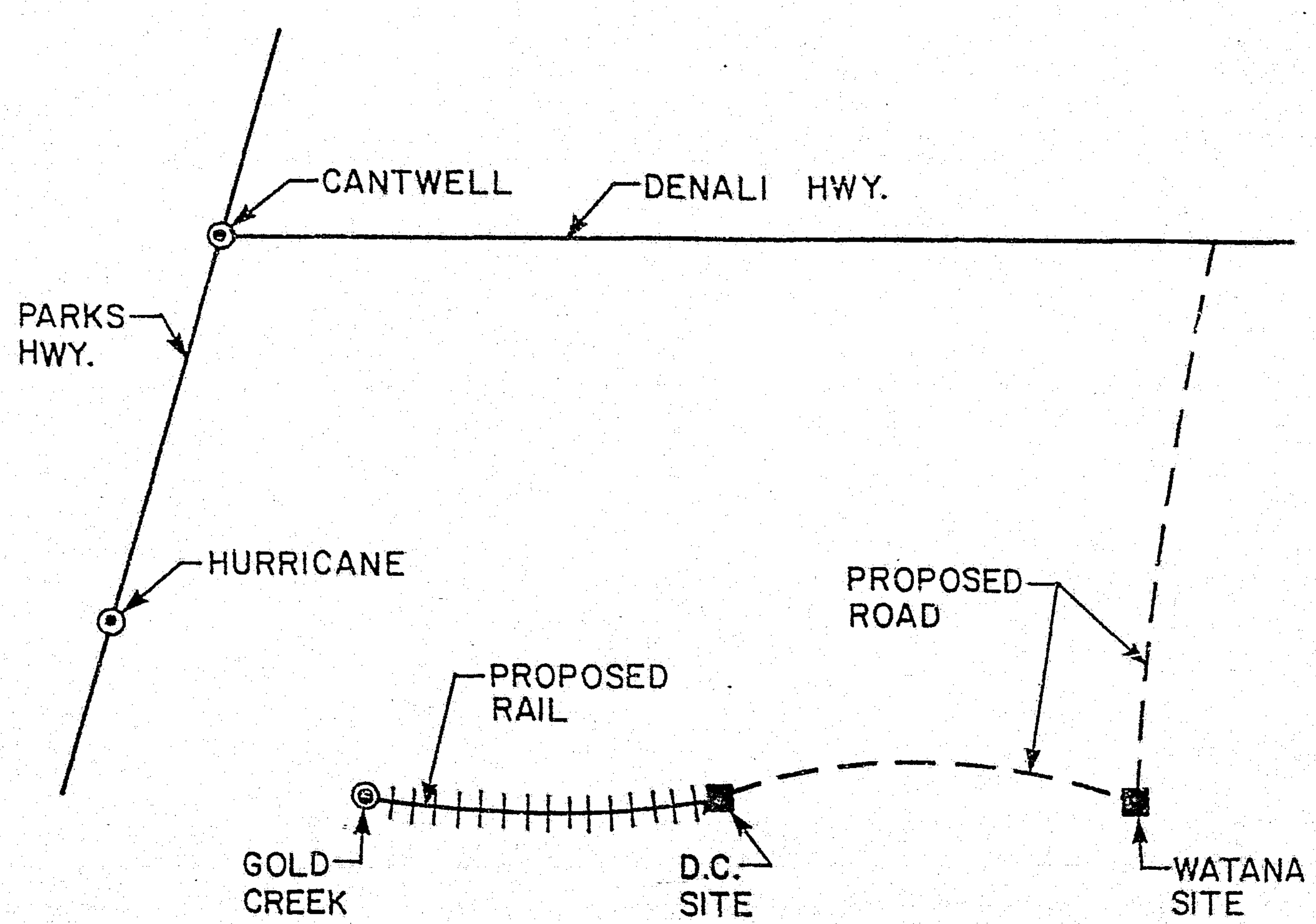
PLAN 4

FIGURE 11.3

ACRES



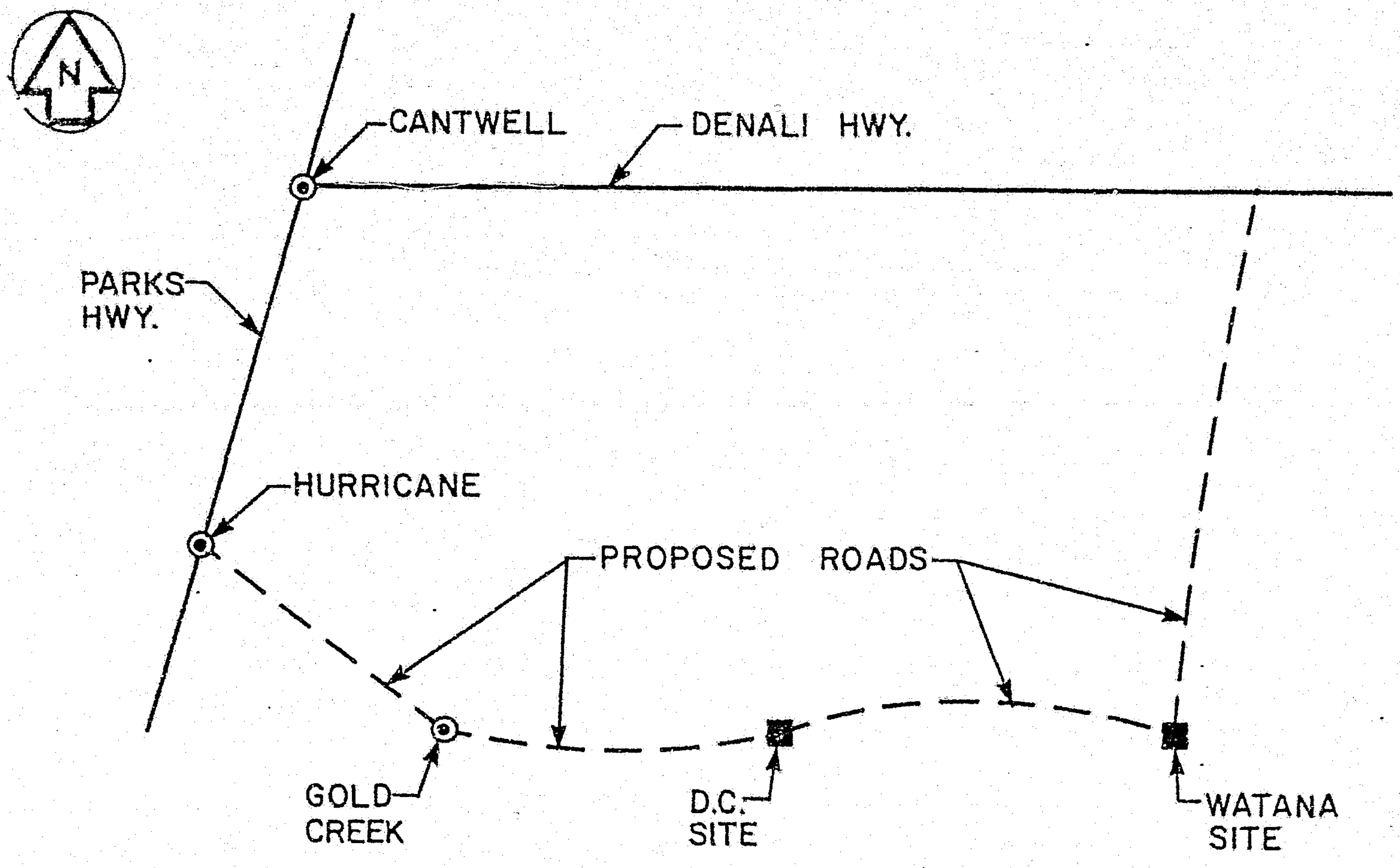
PLAN 5



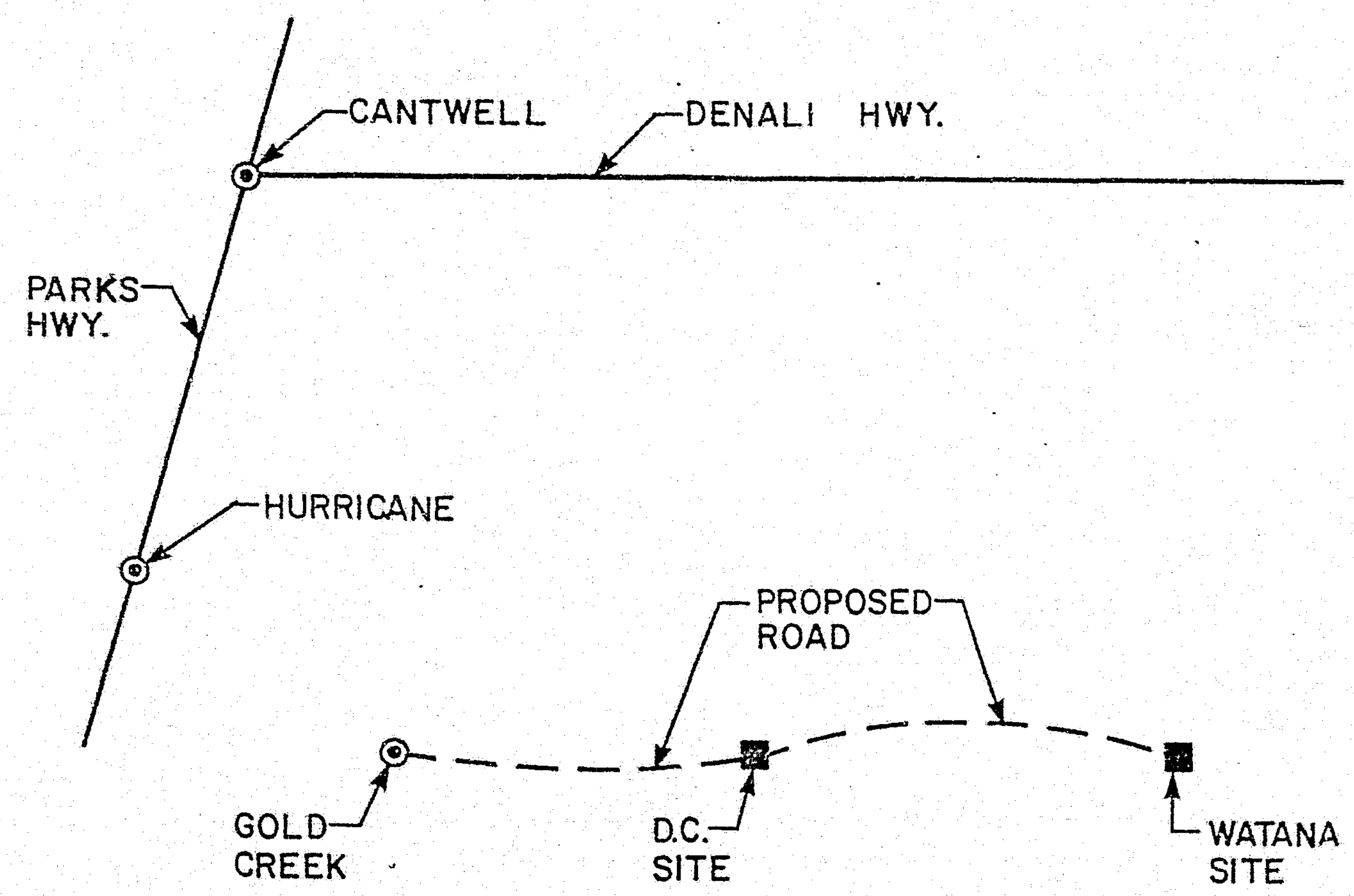
PLAN 6

FIGURE 11.4

ACRES



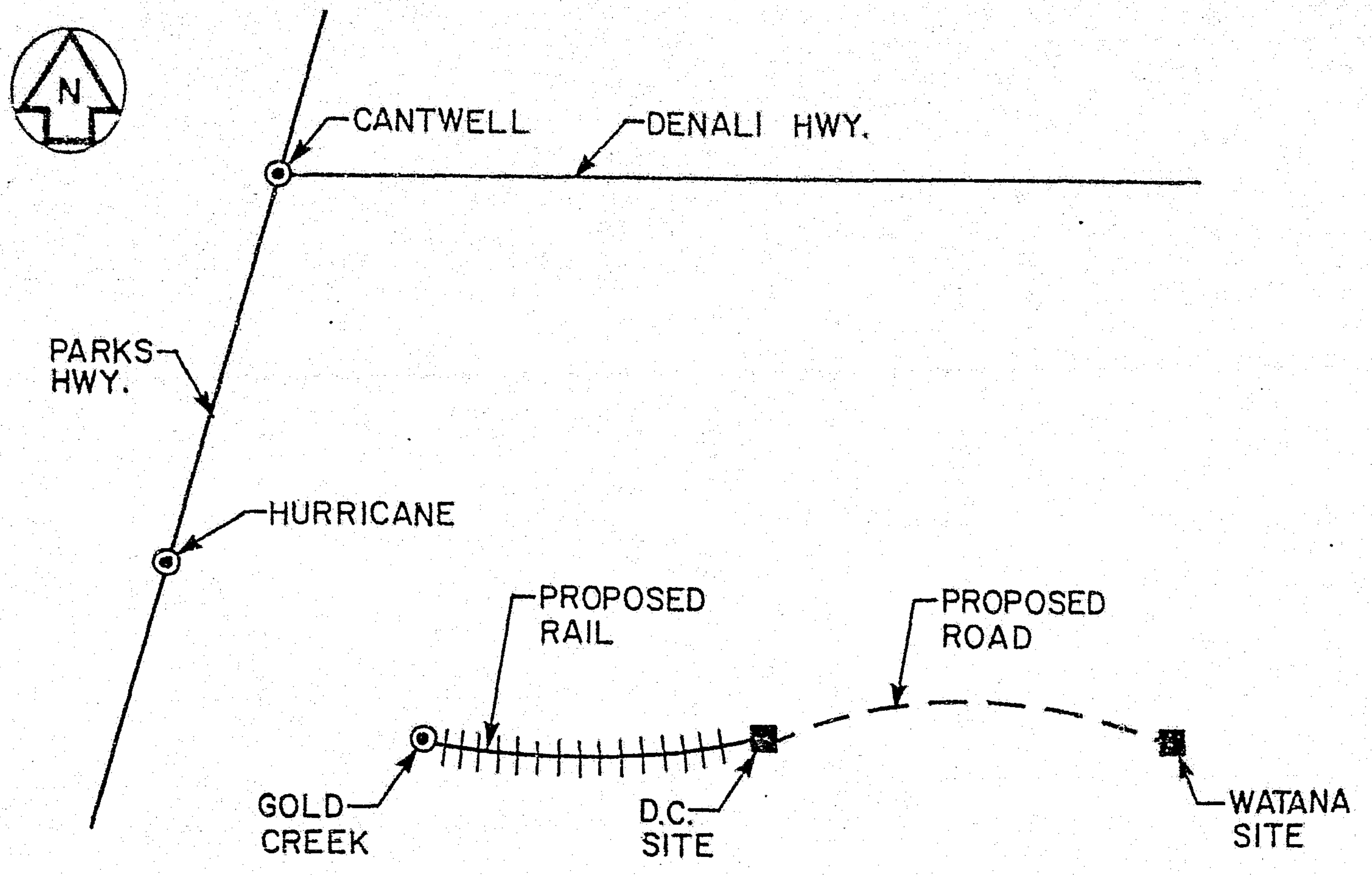
PLAN 7



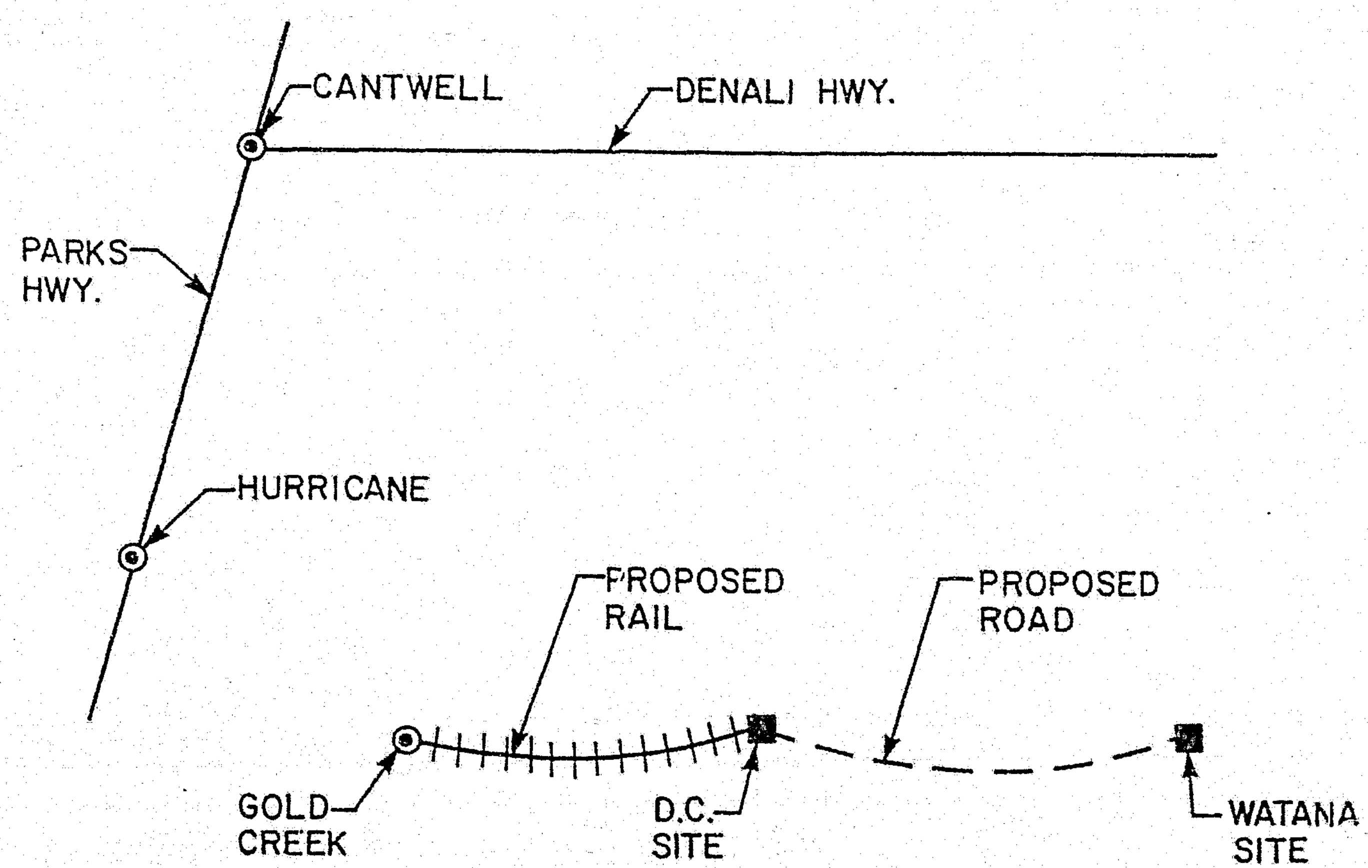
PLAN 8

FIGURE 11.5

ACRES

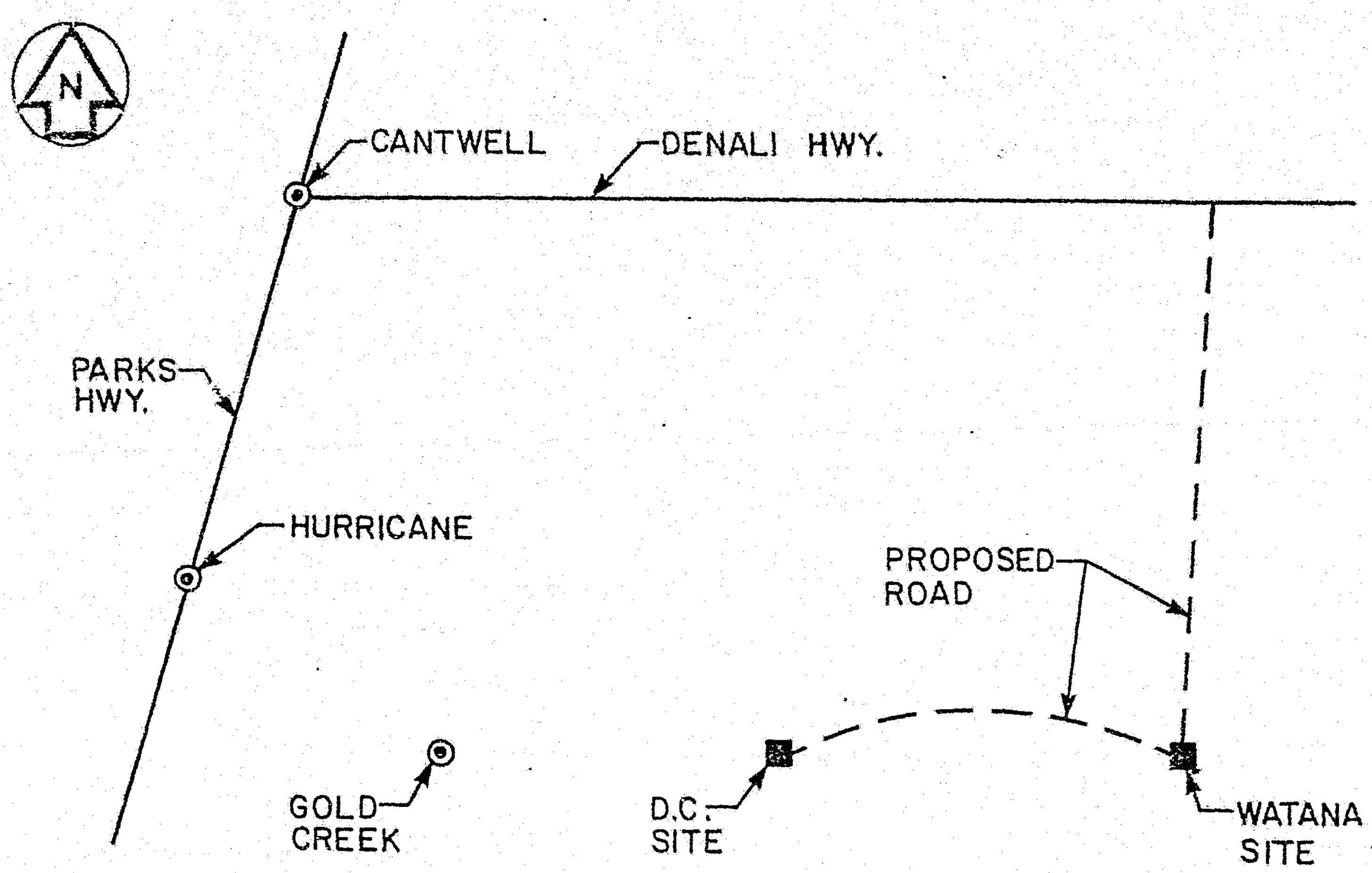


PLAN 9



PLAN 10

FIGURE 11.6



PLAN II

FIGURE 11.7

ACRES

12 - WATANA DEVELOPMENT

This section describes the various components of the Watana Development, including diversion facilities, emergency release facilities, the main dam, primary outlet facilities, reservoir, main and emergency spillways, the power intake, penstocks and powerhouse complex including turbines, generators, mechanical and electrical equipment, switchyard structures, and equipment and project lands. A description of permanent and temporary access and support facilities is also included.

12.1 - General Arrangement

The evolution of the Watana general arrangement is described in Section 9. The Watana reservoir and surrounding area is shown in Plate 31. The site layout in relation to main access facilities, borrow areas and camp facilities is shown in Plate 32. A more detailed arrangement of the various site structures is presented in Plate 32A.

The Watana dam will form a reservoir approximately 48 miles long, with a surface area of 38,000 acres, and a total volume of 9,515,000 acre-feet at a normal maximum operating elevation of 2185. During operation, the reservoir will be capable of being drawndown to a minimum elevation of 2045.

The dam will be an earthfill structure with a vertical central impervious core. The crest elevation of the dam will be 2210, with a maximum height of 885 feet and a crest length of 4,100 feet. The total volume of the structure will be approximately 62,000,000 cubic yards. During construction, the river will be diverted around the main construction area by means of two concrete-lined diversion tunnels, each 40 feet in diameter, on the right bank of the river.

A power intake located on the right bank will comprise an approach channel in rock leading to a multi-level reinforced concrete gated intake structure capable of operation over the full drawdown range. From the intake structure, six penstocks, consisting of concrete-lined tunnels, each 17 feet in diameter, will lead to an underground powerhouse complex housing six Francis turbines with a rated capacity of 170 MW and six semi-umbrella type generators each rated at 180 MVA. Access to the powerhouse complex will be by means of an unlined access tunnel. Turbine discharge will be conducted through six draft tube tunnels to two surge chambers downstream of the powerhouse, then by means of two 30-foot diameter concrete-lined tailrace tunnels. A separate transformer gallery or chamber just upstream from the powerhouse cavern will house nine single-phase 15/345 kv transformers. The transformers will be connected by 345 kV single-phase, oil-filled cable through two cable shafts to the switchyard at the surface.

A tunnel spillway located on the right bank will be designed to discharge all flows resulting from floods having a return frequency of 1:50 years or less. This structure will be equipped with six fixed-cone valves at the downstream end to minimize undesirable nitrogen supersaturation in the river downstream from the dam during spillway operations. Flows resulting from floods with a frequency greater than 1:50 years but less than 1:10,000 years will be discharged by a chute spillway also on the right bank. The spillway control structure at the upstream end will be controlled by three fixed wheel gates leading to a reinforced concrete-line chute section and then to a flip bucket at the downstream end. An emergency spillway on the right bank will provide sufficient additional capacity to permit discharge of the PMF without overtopping the dam. An emergency release facility will allow lowering of the reservoir over a period of time to permit emergency inspection or repair.

12.2 - Site Access

(a) Roads

At Watana the main access road will enter the site from the north. In addition to the main access, several additional roads will be required to the camp, village, airstrip, tank farm, haul roads to the borrow areas, and construction roads to the dam and all major structures. These roads with the exception of the haul roads are shown on Plate 35.

The construction roads will be 40-foot wide gravel surfaced roads with small radius curves and grades limited to 10 percent. Major cut and fill work will be avoided. A gravel pad approximately 5 feet thick will be required for the roads. This gravel pad will provide a drivable surface and also will protect against the sporadic permafrost areas.

(b) Bridges

No major temporary bridges will be required for the construction of the Watana development. The crests widths of the upstream and downstream cofferdams will be planned to provide suitable access to the south bank of the Susitna River during construction.

The completed main dam crest will provide permanent access across the Susitna River.

One area which may require a small temporary bridge is Tsusena Creek near its confluence with the Susitna River. Currently it is envisioned that this crossing can be accomplished with one large or multiple culverts.

(c) Airstrip

A permanent airstrip will be constructed approximately 2.5 miles north of the main construction camp (see Plate 35). The runway will be 6,000 feet in length and will be capable of accommodating the C-130 Hercules aircraft, as well as small jet passenger aircraft. A road will serve the airstrip connecting to the camp, village, and damsite. A small building will be constructed to serve as a terminal and tower and a fuel truck/maintenance facility will be constructed.

A temporary airstrip will also be constructed to support the early phases of mobilization and construction. This temporary runway will be 2,500 feet in length and will be located in the vicinity of the main construction camp. The airstrip will be capable of supporting other type aircraft.

The temporary airstrip will eventually be incorporated into one of the main haul roads for Borrow Area D. This will occur after the permanent airstrip is in service.

(d) Access Tunnel

An access tunnel will be provided to the underground powerhouse and associated works. The main access tunnel will be approximately 35 feet wide and 28 feet high. The tunnel will allow permanent access to the operating development and will also be utilized during construction as the main construction tunnel. Construction adits will branch off to the various components of the development during construction.

12.3 - Site Facilities

(a) General

The construction of the Watana development will require various facilities to support the construction activities throughout the entire construction period. Following construction, the operation of Watana will require certain facilities to support the permanent operation and maintenance of the power facility.

The most significant item among the site facilities will be a combination camp and village that will be constructed and maintained at the project site. The camp/village will be largely a self-sufficient community housing and maintaining living facilities for 6,000 people during construction of the project. After construction is complete, it is planned to dismantle and demobilize the facility and to reclaim the area. It is additionally planned to utilize dismantled buildings and other items in the camp/village as much as possible for use during construction of the Devil Canyon development. Other site facilities include contractors' work areas, site power, services, and communications. Items such as power and communications will be required for construction operations independent of camp operations. The same will be true regarding a hospital or first aid room.

Permanent facilities required will include a permanent town or small community for approximately 130 staff members and their families. Other permanent facility items will include a maintenance building for use during subsequent operation of the power plant.

A conceptual plan for the permanent town has been developed, however, it is recommended that preliminary design and final design be defered until near the end of construction when more information as to the physical parameters of design is available and, more importantly, the human requirements and preferences are better defined.

Fuel oil has been selected as the means of heating the camp/village facilities.

(b) Temporary Camp and Village

The proposed location of the camp and village will be on the north bank of the Susitna River between Deadman and Tsusena Creek, approximately 2.5 miles northeast of the Watana Dam. The north side of the Susitna was chosen because the main access will be from the north and south-facing slopes can be used for siting the structures and the location. The location is shown in Plate 35.

The camp will consist of portable woodframe dormitories for bachelors with modular mess halls, recreational buildings, bank, post office, fire station, warehouses, hospital, offices, etc. The camp will be a single status camp for approximately 5,000 workers.

The village, accommodating approximately 550 families, will be grouped around a service core containing a school, gymnasium, stores, and recreation area.

The village and camp areas will be separated by approximately 1.5 miles to provide a buffer zone between areas. The hospital will serve both the main camp and village.

The camp location will separate living areas from the work areas by a mile or more and keep travel time to work to less than 15 minutes for most personnel.

The camp/village will be constructed in stages to accommodate the peak work force as presented in Table 12.1. The facilities have been designed for the peak work force plus 10 percent for turnover. The turnover will include allowances for overlap of workers, vacations, and visitors. The conceptual layouts for the camp and village are presented on Plate 36 and 37.

(i) Site Preparation

Both the camp and the village areas will be cleared in select areas for topsoil, and the topsoil will be stockpiled for future use in reclamation operations. At the village site, selected areas will be left with trees and natural vegetation intact.

Both the main camp and the village site have been selected to provide well-drained land with natural slopes of 2 to 3 percent.

A granular pad varying in thickness from 1 to 8 feet will be placed at the main camp, covering most of the areas inside the perimeter fence. This will provide a uniform working surface for erection of the high density housing and service buildings and will serve in certain areas to protect the permafrost where it underlies the camp. In the village area, a granular pad will be installed only as necessary to support the housing units and to provide a suitable base for construction of the temporary village center buildings.

All roadways within the camp/village areas will be flanked by roadside ditches, with culverts carrying water across the intersections. In general, drainage will be through construction of a surface network of ditches. Peripheral ditches will intercept overland flows from adjacent non-cleared land and carry them around the camps.

Runoff will ultimately be directed to existing drainage channels leading to Tsusena Creek for the village, and the Susitna River for the main camp.

(ii) Facilities

Construction camp buildings will consist largely of trailer-type factory-built modules assembled at site to provide the various facilities required. The modules will be fabricated complete with heating, lighting and plumbing services, interior finishes, furnishings, and equipment. Trailer modules will be supported on timber cribbing or blocking approximately 2 feet above grade. Larger structures such as the central utilities building, warehouses and hospital will be pre-engineered, steel-framed structures with metal cladding.

The larger structures will be erected on concrete-slab foundations. The slab will be cast on a non-frost susceptible layer at least the thickness of the annual freeze/thaw layer.

Permawalks will connect the majority of the buildings and dormitories. The permawalks will be heated.

The various buildings in the camp are identified on Plate 37.

A detailed description of the nature and function of the buildings is presented in Appendix D8.

(c) Permanent Town

The permanent town will be located at the north end of the temporary village (see Plate 35) and be arranged around a small lake for aesthetic purposes.

The permanent town will consist of permanently constructed buildings and not factory built prefabricated type modules. The various buildings in the permanent town are listed below:

- Single family dwellings;
- Multifamily dwellings;
- Hospital;
- School;
- Fire station;
- A town center will be constructed and will contain the following:
  . a recreation center
  . a gymnasium and swimming pool
  . a shopping center

The concept of building the permanent town at the beginning of the construction period and using it as part of the temporary village was considered. This concept was not adopted, since its intended occupancy and use is a minimum of 10 years away, and the requirements and preferences of the potential occupants cannot be predicted with any degree of accuracy.

(d) Site Power and Utilities

(i) Power

Electrical power will be required to maintain the camp/village and construction activities. A 345 kV transmission line will be constructed and will service the site from 1987 onward. The 345 kV transmission line will be operated at 138 kV while it is bringing in power to the site. After the Watana development is complete and in operation, the transmission line will supply power to the Intertie from Watana and will operate at 345 kV. Since the transmission line will be required after construction is complete, the only cost of the line attributable to the camps will be the interest costs involved in constructing the line 6 years earlier than required for permanent operation.

During the first two years of construction (1985 and 1986), the power supply will come from diesel generators. These generators will remain on site after 1987 as standby power supply since site contractors will provide for their own construction power after this time. The peak demand during the peak camp population year is estimated at 13 MW for the camp/village and 7 MW for construction requirements, thus totaling 20 MW of peak demand.

The distribution system in the camp and village construction will be 34.5 kV.

Power for the permanent town will be supplied from the station service system at the power plant.

(ii) Water

The water supply system will provide for potable water and fire protection for the camp and village construction and selected contractor's work areas. The estimated peak population to be served will be 6,800 (5,000 in the camp and 1,750 in the village).

The principal source of water will be Tsusena Creek, with a back up system of wells drawing on ground water. The water will be treated in accordance with the Environmental Protection Agency's (EPA) primary and secondary requirements.

A system of pumps and constructed storage reservoirs will provide the necessary system demand capacity. Distribution will be by ductile iron pipe system contained in utilidors. The utilidors will be plywood box sections, most of which will be integral with the permawalks. The distribution and location of major components of the water supply system are presented in Plate 35. Details of the utilidors are presented in Plate 38.

(iii) Waste Water

A waste water collection and treatment system will serve the camp/village. One treatment plant will serve the camp/village, while gravity flow lines with lift stations will be used to collect the waste water from all of the camp and village facilities. The "in-camp" and "in-village" collection systems will be run through the permawalks and utilidors so that the collection system will be protected from the elements.

At the village, an aerated collection basin will be installed to collect the sewage. The sewage will be pumped from this collection basin through a force main to the sewage treatment plant.

An aerated collection basin will be needed at the village to balance out the highly cyclic waste water flows.

The chemical toilets located around the site will be serviced by sewage trucks, which will discharge directly into the sewage treatment plant.

The sewage treatment system will be a biological system with lagoons. The system will be designed to meet Alaskan state water law secondary treatment standards. The lagoons and system will be modular to allow for phased growth and contraction of the camp/village.

The location of the treatment plant is shown in Plate 37. The location was selected to avoid unnecessary odors in the camp as the winds are from the SE 4 percent of the time, which is considered minimal.

The sewage plant will discharge its treated effluent through a force main to Deadman Creek. All treated sludge will be disposed in a solid waste sanitary landfill.

(e) Contractor's Area

The onsite contractors will require office, shop, and general work areas. Office space for the contractors has been provided and its location is shown on Plate 35.

Partial space required by the contractors for fabrication shops, maintenance shops, storage or warehouses, and work areas within the camp confines has been designated and is shown on Plate 37. Additional space required for the aforementioned items will be located between the main camp and the main access road.

12.4 - Diversion

(a) General

Diversion of the river flow during construction will be accomplished with two 38 foot diameter circular diversion tunnels. The tunnels will be concrete-lined and located on the right bank of the river. The tunnels are 4,050 feet and 4,140 feet in length. The diversion tunnels are shown in plan and profile on Plate 39.

The tunnels are designed to pass a flood with a return frequency of 1:50 years, equivalent to peak inflow of 81,100 cfs. Routing effects are small and the peak flow the tunnels will discharge a peak flow of 80,500 cfs. The maximum water surface elevation upstream of the cofferdam is EL 1536. A rating curve is presented in Figure 12.1.

The upper tunnel or tunnel No. 2 will be converted to a permanent low level outlet after construction. The local enlarging of the tunnel diameter to 45 feet is to accommodate the low level outlet regulating devices after the tunnel is used as a diversion tunnel.

(b) Cofferdams

As discussed in Section 9 the nature and riverbed will require a slurry wall through the alluvium material to bedrock. The soil/bentonite slurry wall will be constructed through the closure dam and alluvium material to bedrock in order to minimize the amount of seepage into the main dam excavation. The abutment areas above riverbed will be cleared and grubbed, with excavation of all overburden material to sound rock prior to placement of any cofferdam fill.

The upstream cofferdam will be a zoned embankment founded on the closure dam (see Plate 40). The closure dam will be constructed to Elevation 1475 based on a low water Elevation 1470, and will consist of coarse material on the upstream side grading to finer material on the downstream side. When the closure dam is completed the soil/bentonite slurry wall will be constructed to minimize seepage into the main dam foundation excavation. A dewatering system will be established in the main dam excavation to control inflow from the abutments and runoff.

Above Elevation 1475 the cofferdam will be a zoned embankment consisting of a central impervious core, fine and coarse upstream and downstream filters, and rock and/or gravel supporting shell zones with rip-rap on the upstream face.

The downstream cofferdam will be a closure dam constructed from approximate Elevation 1440 to 1472. It will consist of coarse material on the downstream side grading to finer material on the upstream side, with a soil/bentonite slurry wall can be constructed in the finer material as described above for the upstream cofferdam.

The upstream cofferdam crest elevation has a 9 foot freeboard allowance. This includes 3 feet for settlement and wave runup and 6 feet for upstream reservoir ice protection. Large chunks of ice will be present in the river during the spring flood. The 6 foot freeboard prevents ice from overtopping the cofferdam, causing damage.

(c) Tunnel Portals and Gates Structures

A reinforced concrete gate structure will be located at the upstream end of each tunnel, housing two closure gates (see Plate 41).

Each gate will be 40 foot high by 16 foot wide separated by a center concrete pier. The gates will be of the fixed roller vertical lift type operated by a wire rope hoist. The gate hoist will be located in an enclosed, heated housing. Provision will be made for heating the gates and gate guides. The gate in Tunnel No. 1 will be designed to operate with the reservoir at elevation 1,540, a 50 foot operating head. The gate in Tunnel No. 2 will be designed to operate with the reservoir at elevation 1,540, a 120 foot operating head. The gate structures for Tunnels No. 1 and 2 will be designated to withstand external (static) heads of 130 and 460 feet, respectively. The downstream portals will be reinforced concrete structures with slots for stoplogs (see Plate ____).

(d) Final Closure and Reservoir Filling

As discussed in Section 9 one of the diversion tunnels will be converted to a low level outlet or emergency release facility during construction.

It is estimated one year will be required to construct and install the permanent low level outlets in the existing diversion Tunnel No. 1. This will require the that lower Tunnel No. 2 pass all flows during this period. The main dam will be at an elevation sufficient to allow a 100 year recurrence period flow (90,000 cfs) to pass through Tunnel No. 2. This flow will result in a reservoir elevation of 1625. During the construction of the low level outlets, the intake operating gate in the upper Tunnel No. 1 will be closed. Prior to commencing operation of the low level outlets, coarse trashracks will be installed in the upstream intake structure in slots provided.

Upon completion of the low level outlet in the upper tunnel the intake gate will be opened and the low level outlets will commence operation. Upon commencing operation of the low level outlets the lower Tunnel No. 2 will be temporarily closed with the intake gates and construction of the permanent plug will also commence upon filling of the reservoir. It is estimated it will take 12 months to completely place and cure the plug.

During this time the main dam crest elevation will have reached an elevation sufficient to start reservoir impounding and have sufficient storage available to store a 250 year recurrence period flood corresponding to a reservoir elevation of 1,890 feet.

During this time the low level outlets will be passing the minimum summer and winter flows of 6,000 cfs and 800 cfs.

The filling of the reservoir will take 4 years to complete to full reservoir operating elevation of 2185. After 3 years of filling the reservoir will be at elevation 2150 and will allow operation of the powerplant to commence.

The filling sequence was determined from the main dam elevation at that time during construction, the starting reservoir pool elevation at that time during construction, and the capability of the reservoir storage to absorb the inflow volume from a 250 year recurrence period inflow without overtopping the main dam. The 250 year recurrence period flood volume was selected to be consistant with the recurrence period flows and risks used for the design of the diversion and entire project.

This information is presented graphically in Figure 12.2.

Once closure of the upper diversion tunnel is made trashracks will be installed at the upstream entrance to the tunnel. The racks will serve to prevent debris from being drawn into the intake and damaging the high pressure slide gates. The trashracks will be a permanent installation with no provision for removal except with the assistance of divers.

The trashracks will have a bar spacing of about 3 feet and will be designed for a maximum differential head of about 40 feet. To limit the maximum net velocity through the trashracks to about 12 ft/s the racks will be semicircular. Provision will be made for the monitoring the head loss across the trashracks.

12.5 - Emergency Release Facilities

The upper diversion Tunnel No. 1 will be converted to a permanent low level outlet, or emergency release facility. These facilities will be used to pass the required minimum discharge during the 4 year reservoir filling period and will also be used for draining the reservoir in an extreme emergency case.

The facility will have a capacity of 30,000 cfs at full reservoir pool and will be capable of drawing the reservoir dam in 14 months. The reservoir drawdown time incorporating the low level outlets presented graphically in Figure 12.2 for various "start" times during the year.

During operation, energy will be desigrated by means of two gated concrete plugs separated by a 340 foot length of tunnel (see Plate 43).

Bonnetted type high pressure slide gates will be installed in the expansion chamber tunnel plugs for the upper diversion tunnel, once closure of the tunnel is made. The gates will be arranged in groups of 3 gates in series, each group consisting of 1 upstream emergency gate and one operating gate in the upstream plug and one operating gate in the downstream plug (see Plate 43).

The slide gates will be 7.5 feet wide and 11.5 feet high and will be of welded steel construction. The gates will be designed to withstand a total static head of about 740 feet, however will only be operated with a maximum head of about

560 feet. To reduce energy dissipation problems, all three groups of gates will be operated only when the head is less 460 feet.

During operation, the operating gate opening in the upstream plug will be equal to the opening of the corresponding gate in the downstream plug. This will effectively balance the head across the gates. The maximum operative head across a gate will not exceed about 320 feet.

Each gate will have a hydraulic cylinder operator designed to raise or lower gate against a maximum head of 560 feet. Three hydraulic units will be installed, one for the emergency gates, one for the upstream operating gates and one for the downstream operating gates. The nominal operating pressure will be 2,000 psi. Each gate will have an opening/closing time of about 30 minutes. A grease system will be installed in each gate for injection of grease between the gate leaf and gate body seats to reduce frictional forces when the gates are operated. Both local and remote operation of the gates will be possible.

As no facilities will be installed for dewatering the area around the emergency slide gate, the design of the gate will be such that the hydraulic cylinder as well as the cylinder packing may be inspected and repaired without dewatering the area around the gate.

An air vent will be installed at the downstream side of the operating gate in the downstream plug. Air will be drawn from the access shaft. The vent will be heated as necessary to ensure that freezing will not occur.

To prevent concrete erosion, the conduits in each of the tunnel plugs will be steel lined.

The upstream gate operating chamber will be separated from the downstream chamber by a water-tight bulkhead door. In the event of a major failure of the upstream emergency gate, water would be contained within the upstream gate- operating chamber.

12.6 - Main Dam

(a) Comparison with Precedent Structures

The main dam at Watana, as currently proposed with a height of 885 feet will be among the highest in the world. The highest embankment dams completed in North America are Mica Creek in British Columbia (794 feet) and Oroville in California (771 feet). Two dams under construction in the USSR will exceed 1,000 feet, but the only dam completed to a height in excess of 800 feet is Sulak in the USSR. A list of embankment (earth and rockfill) dams in excess of 500 feet completed, under construction or proposed is given in Table 12.2.

The Watana site is located in a seismically active area and the major design features of 24 embankment dams between 350 and 795 feet in height constructed in seismic areas are summarized in Table 12.3. The characteristics of the Watana design which will be developed in this section are included in this table for comparison. Special site conditions, depth to bedrock, availability of materials, size of reservoir, site location, for example, all have an impact on the design and such factors account for some of the extremes quoted in the table.

A further comparison is given in Table 12.4 which includes the principal geometrical parameters of the core and outer slopes for high dams in seismically active areas. Considering these various parameter:

- The freeboard ranges between 13 and 62 feet, with seven of the eleven cases quoted being less than the 25 feet proposed for Watana.

- The crest width ranges between 33 and 111 feet. Wide crests are usually the result of non-structural requirements, i.e., a roadway across the dam. Neglecting extreme widths, seven of the ten cases quoted are between 30 and 40 feet compared with the 35-foot width proposed for Watana.

- The core width ratio ranges between 0.29 and 0.56, with only one example higher than the 0.50 ratio proposed for Watana.

- The upstream slopes range between 2.0:1 and 2.7:1. The Japanese dams tend to have flatter slopes (within the range 2.5 to 2.7), while the North American dams are in the range of 2.0 to 2.6. The Watana slope of 2.4:1 is among the steepest, but is flatter than the next two highest dams, Mica at 2.25:1 and Chicoasen at 2.2:1. However, special features are included in the Watana design, primarily the use of free-draining processed gravel in the upstream shell, to minimize the effects of earthquakes on the stability of the upstream face of the dam.

- The downstream slopes range between 1.8:1 and 2.7:1. Ten of the 15 cases quoted are equal to or steeper than the 2:1 slope proposed for Watana, while only one case is flatter than 2.2:1.

Complete details of the core materials used in all the dams listed in the accompanying tables are not available in the literature. However a number of large dams have been constructed in Canada using similar glacial deposits as core material. The mean grading curves for these materials including that used for the core of the Mica Creek dam, the existing dam generally comparable to Watana in size, materials, and location, are compared with the mean grading curve for the core material proposed for the Watana dam in Figure 12.5. It is apparent from this figure that the Watana core material is well within the range of materials used successfully for other large dams in North America.

In summary, the proposed Watana design is generally conservative with respect to precedent design. However, special features which are discussed in more detail later in this section are incorporated in the Watana section to provide additional safeguards against seismic loading.

(b) Excavation and Foundation Preparation - General

The geology of the Watana site is described in Section 9. In summary, the existing conditions at the damsite comprise alluvial deposits in the riverbed up to 80 feet deep overlying bedrock, while the lower slopes of the valley are covered with talus and there is overburden on the upper slopes. The bedrock is jointed and weathered at the surface with weathering along joints extending to considerable depths. Locally in shear zones and drainage gullies the rock is weathered throughout to depths in excess of 40 feet. The frequency of joints and fractures generally decreases with depth but fractured and weather zones have been identified locally at depths up to 200 feet. Zones of permafrost occur, particularly in the south abutment.

The dam foundation must satisfy the following basic requirements:

- The foundation under the core must be stable and capable of supporting the weight of the core under all loading conditions, must not erode under the seepage gradients which will develop under the core, and must not allow excessive seepage losses under the core.

- The foundation under the upstream and downstream shells must be stable and capable of supporting the weight of the dam without excessive movement under all loading conditions.

- The core material must be prevented from moving down into the foundation (e.g. into cracks or joints) and then through the foundation under the transition zone into the downstream shell or beyond.

- The contact between the core and its foundation must remain watertight despite the distortions that will occur in the dam because of its own weight and the thrust of the reservoir.

- Any seepage through the foundation must be controlled and discharged so that excessive seepage pressures do not develop in the downstream portion of the core, in the materials beneath the shell, or downstream of the dam.

The excavation and foundation preparation necessary to meet these objectives are outlined in the following paragraphs.

(c) Excavation

(i) River Excavation

The properties of the river alluvium are not well defined but it includes sands, gravels, cobbles, and boulders up to 3 feet or more. Such materials are not suitable as a foundation for the core, primarily because of their relatively high permeability. Such alluvial deposits have been left in place under both upstream and downstream shells of many of the world's largest dams (see Table 12.3). However, at Watana these granular materials could undergo liquefaction under seismic loading with potentially catastrophic results. Insufficient data is available to demonstrate that there is no possible risk of liquefaction of the alluvium, but further investigations may provide data to support the concept of removing the alluvium only under the central portion of the shells. However, in view of the high seismicity of the area it is proposed that the river alluvium be removed over the whole foundation area.

(ii) Under Core and Filters

The core and filters must be founded on sound rock to ensure that no material can wash through open joints. This will require excavation of overburden and talus on the slopes and weathered rock in the valley bottom and on the abutments. The talus thickness on the abutments perpendicular to the slope varies from zero to an estimated 20 feet and weathered rock to 40 feet or more in some areas. Weathered rock is here defined as closely jointed or fractured rock with weathering and infilling of the joints. The final foundation will be sound hard rock with only minor weathering, which can be grouted to ensure that core material cannot be washed through joints in the rock.

The maximum rock slope along the abutments is determined to some extend by the valley shape. In general, 1H:1V slope or flatter is ideally preferred although steeper slopes have been used. At Watana damsite, the natural slopes at lower elevations are relatively steeper but still generally less than 1H:2V. It is therefore proposed that the overall core foundation slopes will be no steeper than 1H:2V below elevation 1800 and 1H:1V above elevation 1800. The cross cut slopes will be 1H:10V.

Local irregularities in the rock surface are undesirable because of the potential for differential settlements or strains in the core that could cause cracking and potential piping through the core. Such irregularities also make it difficult to compact the core material to form a tight core-rock bond and they must be eliminated either by additional rock excavation or the addition of concrete to achieve an acceptable slope. Such slopes would normally be on the order of 1H:2V in the lower sections of the dam where contact pressures are higher, flattening to 1:1 at higher elevations.

The depth of excavation required to remove unsuitable rock will vary considerably over the core contact area. In some area very little excavation may be needed, while in highly weathered zones excavation may extend to 50 or 60 feet. On the basis of available data, it is estimated that the average excavation under the core and filters will be 40 feet.

(iii) Under Upstream and Downstream Shells

The shells will be founded on competent rock. Loose or detached rock or rock ribs and highly weathered rock will be removed to expose sound rock. Weathering along joints and local irregularities in the rock surface will be acceptable. The actual thickness of rock to be excavated will vary across the site, but it is estimated that the average will not exceed 10 feet.

(iv) Dental Excavation

Dental excavation over and above normal excavation is expected in zones of intense shearing or highly irregular surfaces. Whereas the need for such excavation has been identified by investigations completed to date, the magnitude has not been properly assessed because of heavy vegetation, tundra cover, and general lack of outcrops and access problems.

(v) Excavation Methods

It is expected that the excavation of the overburden material within the dam foundation will be performed as a multi-level operation using wheeled loaders working with dozers. Boulders that cannot be removed by excavation equipment will be blasted. On the steep working areas will be formed with material excavated from the slopes above. These working areas will be progressively lowered removing overburden and weathered rock in one operation.

The excavation of the foundation will need to be complete for safety reasons from about Elevation 1800 down to the riverbed before placing of fill is commenced. The excavation on the upper slopes will then require to be kept sufficiently in advance of grouting and fill placement to avoid interference of these activities by the blasting. Excavation of unrippable weathered rock and trimming of the rock surface to acceptable slopes will require blasting which will restrict other activities such as surface grouting.

Numerous access roads will be required throughout the dam area to reach the various working levels. Dental excavation will be done by small backhoes and final cleanup of the area under the core and filters will be carried out to a high standard by hand with high pressure water jetting prior to grouting. The rock surface under the core and filters should be clean enough for detailed geological mapping and for grout leaks to be observed and caulked if necessary. The foundation must be free from snow and ice before fill material is placed.

Selected alluvial material from the riverbed may be used in the downstream shell of the dam but the remaining material, generally a mixture of weathered rock and overburden, will be wasted or used for road, cofferdam or temporary facility construction. Spoil areas will generally be below final water level in the reservoir area.

(d) Grouting and Pressure Relief

A combination of consolidation grouting and cutoff curtain grouting under the core and a downstream pressure relief (drainage) system are proposed for the Watana site. Those systems will result in:

- Improved stability of the foundation;

- Reduction in rock mass permeability and hence seepage through the foundation;

- Reduction in the risk of movement of soil particles through joints in the rock; and

- Control and safe discharge of any seepage flows through the grout curtain.

It is proposed that the curtain grouting and drilling for the pressure relief system are carried out from galleries in the rock foundation in the abutments and beneath the dam. Details of the grouting, pressure relief and galleries are shown on Plate 46.

The purpose of grouting is to improve foundation and abutment rock conditions with respect to load bearing and seepage considerations. The need, extent, and detail of grouting is dependent on site geological conditions, type, and character of rock, reservoir head, and location of specific structures. The diorite bedrock at Watana is competent as far as load carrying capacity is concerned. However, numerous shear zones from a few inches to several feet in width, have been identified in a general NW-SE direction. Occasionally, the width of shear zones may be several tens of feet locally. Most of these zones, which are found both in the river channel and in abutments, contain gouge material and under appropriate conditions, may be susceptible to piping. These features are discussed in more detail in Section 9.

The permeability tests in boreholes indicate the rock mass permeability at the Watana site to be generally in the range from 1×10^{6} cm/sec to 1×10^{-4} cm/sec, indicating a maximum seepage rate through the foundation of the order of 4 cubic feet per second. However, these permeability values may not properly account for shear zones. For example, in Borehole BH-2 on the north abutment, circulation was lost during drilling when the boring encountered a shear zone. Also, because of heavy vegetation, talus cover and limited access, it is possible that there may be other shear zones not yet identified. A properly conducted primary grouting program of an exploratory nature will be required under the dam and in the abutments and, depending on the results of this program, additional grouting including multiple line curtains may be required.

(i) Consolidation Grouting

The rock under the core, upstream filter, and downstream filter will be consolidation grouted to provide a zone of relatively impermeable rock under the entire contact. Locally, the rock may be sound and free of any discontinuities resulting in virtually no grout take; nevertheless, the joints and shear zones are generally steeply dipping and any particular vertical plane is likely to intersect these zones which are estimated to be 15 to 20 feet apart. Consolidation grouting is estimated to require 30 foot deep holes on a 10 foot by 10 foot grid.

(ii) Curtain Grouting

The design of grout curtains under dams is largely empirical, though based on data from boreholes. At the Watana site, only borehole DH-21 extends to a significant depth below the river to elevation 876 feet, approximately 500 feet below dam foundation level. Sheared and highly fractured zones are indicated at an average of 50 feet intervals to the bottom of the hole and the upper zones should be grouted to reduce seepage losses. The average rock permeability decreases significantly around 200 feet depth. A grout curtain is not expected to be 100 percent effective in eliminating seepage, but rather to increase the length of the seepage path. Flow net analysis indicates that a positive cutoff of 350 feet deep increases the potential flow path by a factor of 1.7, decreasing the average hydraulic gradient from about 0.9 to 0.5.

For the purposes of this study, a double row grout curtain to a depth of 0.7H, where H is the head of water behind the dam at a particular location, with a maximum depth of grout curtain of 350 feet has been assumed. Grouting will be carried out from a series of underground galleries which will also serve the drainage system pressure relief.

It is likely that in some areas the grout take at depth will be very low. Primary holes will be considered as exploratory holes and will be core drilled. On the basis of the core and water pressure tests in the exploratory holes, the depth of secondary holes can be decided. The exploratory holes may also identify areas that need additional grouting.

All holes will be water pressure tested in stages and the grouting program will be determined using these results. Grouting will be carried out using split spacing with the primary holes at 40 feet spacing. The secondary, tertiary and quaternary holes would bring the final hole spacing to 5 feet if required.

Permafrost in the area to be grouted will have to be thawed before water pressure testing and grouting can be done. The greatest depth of permafrost so far recorded was in BH-8 where the hole froze up to

175 feet depth. Additional boreholes may be required for the thawing of sufficient rock to form an effective curtain. Rock will be thawed by circulating clean river water through drill holes.

The effectiveness of the initial thawing and grouting may be difficult to assess but the permanent galleries under the dam will enable additional grouting to be carried out at any time during the following reservoir filling and subsequent thawing of the foundation and abutments.

It is desirable for the grout holes to intersect as many discontinuities as possible. The dip of the main joint sets and shear range from 80° to 60° and its is therefore considered most efficient to drill the boreholes vertically or at an angle of 45°.

A major shear zone approximately 600 feet wide trending in a NW direction intersects the left edge of the dam and reservoir area and the curtain should extend into the abutment to provide a positive cutoff of this zone. The ground surface rises to the south of the dam and the surface expression of any shears to the south of the major zone will be outside the reservoir area and are unlikely to cause appreciable seepage.

The extent of the grout curtain through the shear zone will be determined by exploration from the grout gallery.

Artesian water pressure was observed at the shear zone in BH-12 indicating that materials with high permeabilities exist in the shear zone which must be effectively grouted.

No major shears have been found on the right abutment where the rock is of good quality. The grout curtain will extend from the spillway intake structure 400 feet into the abutment with the depth of the curtain set at a minimum of 200 feet.

The spillway control structure is located on the dam centerline and the grout curtain will extend beneath the structure with drilling and grouting from the gallery formed within the concrete rollway.

Drainage will be provided behind the grout curtain with holes drilled from the gallery.

(iii) Drainage and Pressure Relief

Drainage features are included beneath the dam foundation and the abutments to intercept seepage through the grout curtain and relieve pressure. Common drainage and grouting will be constructed with grouting from the upstream side and drainage from the downstream side of the galleries. The use of galleries is recommended for the following reasons:

- Curtain grouting from the gallery can be carried out independently of the construction of the dam. This can shorten construction times.

- The grouting can continue longer into the winter than would have been possible with surface grouting.

- Permanent access is available under the dam for inspection. Additional grouting or drainage holes may be drilled after construction of the dam which is an important consideration where there is permafrost. The thawing effect of the reservoir may require remedial grouting after impounding has commenced.

- Higher grout pressures can be used if required because of the overlying weight of embankment.

- Drainage holes drilled downstream of the grout curtain will be discharged into the gallery enabling flow from individual holes to be monitored. This system will prevent the outlets of the drainage holes freezing which is an essential requirement. Gallery drainage is more effective because pressures are relieved at a lower level.

- The galleries may be used for the installation of instrumentation and provide access for the repair and replacement of instrumentation.

- Tunneled galleries provide the great advantage as an exploration tunnel for the rock of the dam foundation. The tunnel gives the best opportunity for understanding the nature of the rock along the grout and drainage curtains which will be invaluable in the faulted and sheared zones.

The drainage/pressure relief holes will be drilled after all grouting is complete. They will be 3 inches in diameter spaced at approximately 10 foot centers. Generally the holes will be open but any penetrating fractured or sheared rock may require perforated casing to prevent caving.

(iv) <u>Construction Methods - Grouting and Pressure Relief</u>

The schedule of work is of particular importance in this phase of the work. The excavation for the galleries must be carried out before consolidated grouting because the grouted rock may be disturbed by the blasting for excavation. It will also be preferable to complete excavation of the dam foundation in a particular section before excavation of the gallery so that the surface rock profile may be confirmed before tunnelling.

Rock temperatures will be measured any any areas of permafrost thawed prior to grouting. Grout holes will generally be 1-1/2 inch in diameter. Large hole sizes will be drilled where exploratory cored holes are required or down-the-hole hammer equipment is used.

All holes will be washed and pressure tested before grouting. Grouting will be done with Type II Portland cement with 2 percent addition of bentonite (by weight of cement). The water/cement ratio and grouting pressures will be varied according to the conditions encountered. Grouting will be carried out in stages using packers. Some redrilling between stages will be required. To allow greatest flexibility of the schedule, most curtain grouting, which will include up-hole grouting will be done from the galleries. In the inclined galleries special platforms will be required for drilling and grouting equipment.

Primary grout holes will be treated as exploratory holes and core drilled with further core drilled holes as required to test the effectiveness of the grouting. The grouting program will be modified according to the rock conditions encountered as the work proceeds.

(v) Gallery Construction

The layout of the galleries are shown on Plate 46. The horizontal and inclined tunnels will be excavated by conventional drill and blast methods. Vertical shafts will be raise bored providing a smooth excavated profile with little support required. It is expected that the majority of the gallery length will not require any support but from available geologic data it is estimated that about 25 percent will require rock bolt and shotcrete support. Steel arches will be required at the portals and at tunnel junctions or in highly fractured or sheared zones. A concrete slab will be cast in the tunnel invert to provide an even working surface and to form the drainage channel.

Measuring weirs will be constructed in the drainage channels in order that the volume of sewage water may be monitored. The seepage water will be discharged from the gallery just above tailwater level through drainage tunnels extending to the downstream toe of the dam. The drainage outlet of these tunnels will be located under tailwater level to prevent icing up of the outlet. Inspection access will be provided at the downstream toe of the dam but from a separate portal above water level.

Lighting for inspection of the galleries and ventilation will be required. The fresh air intake during the winter must be heated to prevent freezing of seepage water within the tunnels. The ventilation will only be required occasionally when personnel are in the tunnels. Elevators will be installed in the vertical shafts together with emergency stairs and cable hoists installed in the inclined tunnels for movement of equipment.

(e) Main Dam Embankment

The main dam will consist of a central compacted core protected by fine and coarse filters on the upstream and downstream slopes. The downstream outer shell will consist of rock fill and alluvium gravel; and the upstream outer shell of clean alluvium gravel. A typical cross section is shown on Plate 45.

(i) Comparison of Vertical and Inclined Cores

The design of the embankment is dependent on the type of core chosen, either a vertical core or an inclined core, and its location, upstream or central in the embankment.

The advantages to each type of core are as follows:

- Vertical Core

Provides better contact with the foundation;

Provides slightly more thickness of core for the same quantity of the core material; and

Settlement of the core will be independent of the post-construction or seismic displacement settlement of the downstream shell.

- Inclined Core

Can place bulk quantity of downstream shell before placing core material; and

Can carry out foundation treatment during placement of shell material.

The major disadvantages for each type of core are as follows:

- Vertical Core

Placement of core material controls placement of filters and shell materials; and

Possible arching of a thin core by transferring weight to adjacent filters and shell materials during settlement or seismic displacements.

- Inclined Core

Excessive post-construction settlement or seismic displacment of downstream shell may cause rupture of core; and

Location of core may effect upstream slope by making it flatter for stability reasons.

A central vertical core was chosen for the embankment based on a review of precedent structures discussed above and the nature of the proposed impervious material.

The proposed impervious material is a combination of glacial outwash and tills with a wide grain size distribution. This material is nonplastic and would tend to crack rather than deform under tensile stress and hence may be susceptible to erosion. For a sloping core the possibility exists of cracks developing in the core for a nonplastic material because of lateral settlement or displacement during a seismic event. It also becomes difficult to avoid high tensile and shearing stresses in an inclined core. Settlement data indicates that the magnitude of water load settlements in rockfill dams may increase at a rate greater than direct proportion to the height of the dam. For these reasons a central vertical core will be used in the Watana Dam cross section.

(ii) Earthquake Resistance Design Features

Because of the apparent low plasticity of the material to be used in the impervious core and the requirement for an earthquake resistant design, the following design features will be incorporated into the main dam cross section:

- The core-foundation contact will be widened near the ends of the embankment to ensure seepage control during normal operating conditions and any seismic event.

- Thick filter zones will be placed upstream and downstream from the impervious core to prevent breaching of the core from either post-construction settlement and cracking or from any cracking resulting from a seismic event.

- The filters will be designed to be self-healing in case of transverse cracks in the core resulting from either post-construction settlement or a seismic event.

- The downstream filters will be designed to be capable of handling any abnormal flows which could result from transverse cracking at the core from post-construction settlement or a seismic event.

- The proposed width of the core will prevent arching of the core caused by transfer of load to the shell or filter materials.

- Compacted river alluvium gravel and rock fill will be used to construct the downstream outershell. Compacted processed clean river alluvium gravel of high permeability will be used to construct the upstream outershell to minimize settlement displacement and the build up of pore pressures during a seismic event and to ensure rapid dissipation of any pressures which may occur.

(iii) Freeboard and Static Settlement

The minimum required crest elevation of the Watana Dam, not including static and seismic settlement, was determined for each of the following conditions:

	1:50 Year Storm	1:10,000 Year Storm	Probable Maximum Flood
Normal maximum reservoir elevation	2185	2185	2185
Storm surcharge	6	8	17
Water elevation	2191	2193	2202
Wave runup allowance	6	6	-
Dry freeboard allowance	3	-	-
Elevation top of core	2200	2199	2202
Roadway thickness	3	3	3
Minimum crest elevation	2203	2202	2205

These elevations refer to the maximum section of the dam and are based on a normal operating reservoir level of 2185 feet. The governing minimum crest elevation excluding static and seismic settlement is 2205 feet at the maximum section and at the abutments.

This is the lowest elevation acceptable for the dam crest and allowances must be made for static settlement of the dam following its completion, settlement on saturation of the upstream shell, and possible slumping because of seismic loading. It has not been possible to perform detailed calculations at this time to determine the likely settlements since no test data is available.

For purposes of this feasibility study, it has been assumed that seismic slumping could be up to 0.5 percent of the height of the dam and the design crest elevation at the center of the dam is, therefore, shown at 2210 feet, 5 feet above the required minimum elevation of 2205 feet. An allowance of 2 feet has been made at both abutments and hence the design crest elevation rises from 2207 at the abutments to 2210 feet at the center. Thus, under normal operating conditions the minimum freeboard relative to the maximum operating pool elevation of 2185 will be 22 feet at the abutments and 25 feet at the center of the dam.

If for any reason the crest settles below 2210 feet, more material should be added to maintain the safety margin of 5 feet to allow for seismic slumping.

An additional allowance must also be made for post-construction settlement of the dam under its own weight and because of the effects of saturation on the upstream gravel fill when the reservoir is first filled. This allowance is not shown on the drawings since it is not a permanent requirement. However, for initial estimating purposed, 1 percent of the height of the dam has been allowed. Hence, at the end of construction the dam crest at the center of the dam would be at 2,210 feet plus 9, or 2,219 feet. The additional height constructed into the dam would be achieved by steepening both slopes above approximately elevation 1850.

Further margin against overtopping of the main dam is provided by the emergency spillway. Under normal operation this spillway is sealed by a fuse plug or dam across the entrance channel. This plug is simply a gravel dam with special design of the core and strict control of the shell materials to ensure that it will erode rapidly when overtopped, allowing flood flows to be discharged freely down the emergency spillway.

The location and typical cross section through the fuse plug are shown on Plate 53. The fuse plug has a total length of 310 feet and a height of 31.5 feet. A pilot channel 20 feet wide with an invert 1.5 feet lower than the crest, is provided at the center of the fuse plug, to start the washout at a predesignated location. The location of this pilot channel is chosen so that the flow of water during washout will be smoothly channeled into the spillway chute.

(iv) Typical Cross Section

The typical cross section of the main dam is shown in Plate 45. The central impervious core slopes are 1H:4V with a crest width of 35 feet. The thickness of the core at any section will be slightly more than 0.5 times the head of water at that section. Minimum core-foundation contact will be 50 feet requiring flaring of the cross section at each end of the embankment.

The upstream and downstream filter zones increase in thickness from 15 or 30 feet near the crest of the dam to a maximum in excess of 60 feet. They are sized to provide protection against possible piping through transverse cracks in the core that could occur because of settlement or resulting from displacement during a seismic event.

The shells of the dam will consist of compacted alluvium gravels. To minimize pore pressure generation and ensure rapid dissipation during a seismic event, the saturated upstream shell will consist of compacted clean alluvial gravels processed to remove fines so that not more than 10 percent of the materials is less than 3/8-inch in size. The downstream shell will consist of compacted unprocessed alluvial gravels and rockfill from the excavations for underground work since it will not be effected by pore pressure generation during a seismic event.

Slope protection on the upstream slope will consist of a 10 foot zone of oversize material up to 24 inches in diameter, placed and compacted by suitable equipment.

The typical crest detail in shown in Plate 45. Because of the narrowing of the crest dam, the filter zones are reduced in width and the upstream and downstream coarse filters are eliminated. A layer of filter fabric is incorporated to protect the core material from damage from frost penetration and dessication, and to act as a coarse filter where required.

(v) Core Material Properties

The core material will be obtained from Borrow Area D, located on the right bank of the river, upstream from the dam. The area consists of a series of glacial deposits separated by alluvial and lacustrine deposits. A generalized surficial stratigraphic column in Area D, based on all investigations to date including seismic lines and deep drilling, is given in Table 12.5.

Typical grading curves for each unit are presented in Figures 12.6 through 12.10, which also indicate the range and average moisture content of each unit. It is proposed to blend material from the various units as required to provide core material with a maximum particle size of 6 inches and within specified limits of moisture content, gradation (as shown in Figure 12.11) and plasticity. The composite gradation curve from Borrow Area D is shown in Figure 12.12.

The Atterberg limits will be within the following ranges:

- Plasticity Index - 0 to 20
- Liquid Limit - 10 to 45

Permeability tests indicate a permeability on the order of 10^{5} cm/sec, which is within the normal range for glacial deposits used in similar dams.

Modified Proctor Compaction tests on material passing 3/4-inch sieve indicate an optimum moisture content of 7.5 percent with a maximum dry density of 135.5 pcf. Standard Proctor compaction test results on material passing No. 4 sieve indicate optimum moisture content of 10.4 percent with a maximum dry density of 127.6 pcf. The test results are plotted in Figures 12.13 and 12.14.

The natural water contents of samples tested range from 7 to 21 percent with occasional samples of finer grained material up to 40 percent. Blending and processing of the core material will be necessary while pockets and layers of very wet material will be left in the borrow areas or otherwise wasted.

Consolidated undrained test results (see Figure 12.15) at 95 percent Modified Proctor density and 2 percent above optimum moisture content, indicate the angle of shear strength resistance (Ø) equal to 37°, with a cohesion intercept (c) of zero.

Pinhole dispersion tests indicate that the proposed core material is a non-dispersive material.

Consolidation tests indicate a compression indec (C_c) of 0.06 and test results are shown in Figure 12.16.

(vi) Excavation, Placement, and Compaction of Core Material

The borrow area will be excavated to a depth of approximately 30 feet working vertical faces. Processing and blending of the material will be done during excavation. Oversize material (greater than 6 inches) will be removed by grizzlies or raked out of the fill during spreading. Frozen material will have to be left in place or loosened by blasting and ripping for haulage to waste area. Moisture conditioning will be done in the borrow area.

Material will be placed in 8-inch compacted lifts at a maximum moisture content of 3 percent above optimum moisture content, and compacted to 95 percent of the maximum density obtained from the Standard Proctor test. Type of roller, number of passes, thickness of lift and moisture content can be adjusted based on field tests and equipment to be used.

(vii) Fine and Coarse Filter Materials

Fine and coarse filter material will be obtained from Borrow Area E. The material will be processed to provide the required gradations. Frozen material will, where possible, be allowed to progressively thaw insitu, with a system of surface ditches to accelerate drainage of the thawed material. Where this is not practical for scheduling reasons or other considerations, the foreign material will be loosened by ripping or blasting and hauled to a disposal area. Moisture conditioning will be done in the borrow area.

Criterion 1: The 15 percent (D15) of a filter material must be not more than five times the 85 percent size (D85) of a protected soil.

Criterion 2: The 15 percent size (D15) of a filter material should be at least five times the 15 percent size (D15) of a protected soil.

Criterion 3: The 50 percent size (D50) of a filter material must be not more than 25 times the 50 percent size (D50) of a protected soil.

The required gradations of the fine and coarse filter material to satisfy the above criteria are show[illegible] Figure 12.11, while composite gradations for Borrow Area E a[illegible]own in Figure 12.16.

Permeability of the fine filter and [illegible]rse filter is estimated to be greater than 1 cm/sec and 10 cm/sec, respectively. Permeability will be verified by large scale field or laboratory tests.

The fine and coarse filter material are assumed to have an angle of shearing resistance (Ø) of 35° for the purposes of these studies.

Actual properties will be determined from large scale triaxial tests and/or modeling the gradation for standard triaxial tests for final design.

(viii) Excavation, Placement, and Compaction of Filter Material

The borrow areas will be developed utilizing scrapers and draglines which will supply the required amounts of fine and coarse filter material construction. Material will be processed by screening and blending using wet screening methods. Oversized material will have to be removed and either used as an aggregate source or possibly used in the outershell of the dam.

The method of placement and compaction will depend on the results of full scale test fills to be done prior to construction using the proposed equipment and materials. It is assumed that 12-inch lifts with four passes of a large vibratory roller will provide the required compaction.

(ix) Alluvial Fill Material

The alluvial fill will be obtained from Borrow Areas E and I. The upstream shell of the dam will be constructed using processed alluvial gravel and the downstream shell of unprocessed alluvium fill material mixed with rock from the various excavations, when available. Any oversized material (greater than 18 inches) will be either used in the rip-rap zones or crushed for concrete aggregate.

The gradation of the available alluvial fill material will be as shown in Figures 12.16 and 12.17, while the required grading limits for the upstream shell are shown in Figure 12.11. The downstream shell material will not require processing. Maximum size of river gravel will be 18 inches in the greatest dimension.

Permeability of the processed alluvial fill is estimated to be greater than 100 cm/sec.

An angle of shearing resistance of Ø equal to 35° has been assumed for the alluvial fill material.

Actual properties will be determined from large scale triaxial tests and/or modeling the gradation for standard triaxial tests for final design.

(x) Excavation, Placement, and Compaction of Alluvial Fill Material

The alluvial fill material will be obtained from the main dam foundation excavation and from downstream from the main dam. Method of excavation will be by scraper operations above the water table and dragline operation below the water table to a maximum depth of 50 feet. The material will have to be processed to remove the undersized and oversize material for the upstream shell.

All material in the shells must be well compacted to minimize post-construction settlement and seismic slumping. The method of placement and compaction will depend on the results of full scale test fills to be done prior to construction using the proposed equipment and material. It is assumed that 24-inch lifts for alluvium fill material with four passes of a large vibratory roller will provide the required compaction.

(xi) Rip-Rap Material

The rip-rap material (slope protection) will be obtained from the oversize material from the various borrow areas, Quarry A and any other rock excavations. The rip-rap material will be placed on the upstream slopes and in certain areas on the downstream slopes of the dam exposed to wave action.

The gradation of the rip-rap material, Figure 12.11, is based on a 6-foot wave height using a nomorgraph, Figure 5-6, from EM1110-2-2300. The maximum size of rip-rap material will be 24 inches. It is assumed that 36-inch lifts for the rip-rap zone with four passes of a large vibratory roller will provide the required compaction.

(f) Stability Analysis

(i) Methodology

Static and dynamic stability analyses have been performed to confirm the stability of the upstream and downstream slopes of the Watana dam. The analyses indicates stable slopes under all conditions for a 2.4 horizontal to 1.0 vertical upstream slope and a 2.0 horizontal to 1.0 vertical downstream slope.

The static analyses have been done using the STABL computer program developed to handle general slope stability problems by adaptation of the Modified Bishop method and a finite element program for static analysis of earth and rockfill dams (FEADAM) to determine the initial stresses in the dam during normal operating conditions. The results and conclusions from both the static and dynamic analyses are given in Appendix D.

The dynamic analyses have been done using the QUAD 4 finite element program which incorporates strain dependent shear modulus and damping parameters. The design earthquake for the dynamic analyses was developed for a Benioff zone event.

An assessment of the static and seismic response of the Watana dam for the static and postulated seismic loading involves the following:

- Finite Element Model

  The finite element model consists of 20 layers of elements with approximately 550 nodes and 520 elements. Different soil parameters as described in the following sections have been chosen for the core, transition material, and the shell material. The transition material will comprie the fine and coarse filter zones.

- Static Analysis

  The slope stability and analyses were done using the STABL computer program for the general solution of slope stability problems by a two-dimensional limiting equilibrium method. The calculation of the factory of safety against instability of a slope is performed by an adaptation of the Modified Bishop method of slices which allows the analysis of trial failure surfaces other than those of a circular slope. Soil properties used in the analysis are given in Table 12.6.

  The following conditions were analyzed:

Condition	Required Minimum Factor of Safety	Calculated Factor of Safety U/S Slope	Calculated Factor of Safety D/S Slope
Construction	1.3	2.2 - 2.2	1.7
Normal Operating	1.5	2.0	1.7
Rapid Drawdown	1.0	1.8 - 2.0	1.7
Normal Operating with Maximum Pool	1.3	2.0 - 2.1	1.7

  The calculated factors of safety as shown in the above table indicate no general slope stability problems under static loading.

  Further analysis, using the finite element program for static analyses of earth and rockfill dams (FEADAM), determined the initial stresses in the dam during normal operating conditions. The program calculates the stresses, strains, and displacements in the dam simulating the actual sequence of construction operations. Appropriate nonlinear and stress- dependent, stress-strain properties for the soils were taken from information compiled in Table 5 in Duncan et al (1980). Table 12.6 presents the values used in the analysis. Two analyses were performed to show the effects of relatively soft vs stiff core material.

- Dynamic Analysis

The dynamic analysis was done using the QUAD 4 computer program. The initial values of shear modulus and damping ratio to be used in the analyses were derived from typical values available in Banerjee et al (1979) and are as follows:

ZONE	K_2	Damping Shear Type Curve
Core Material		
- Soft	90	sand
- Stiff	120	sand
Transition Material	150	sand
Shell Material	180	sand

The design earthquake time history was developed by Woodward-Clyde Consultants and is shown in Figure 12.18. The significant features are as follows:

- Magnitude 8.5 Richter;
- Location 40 kilometers below site (Benioff Zone);
- Maximum acceleration of 0.55g;
- Duration of strong motion - 45 sec; and
- Significant number of cycles - 25.

The preliminary dynamic analysis had peak output values occurring about 24 seconds into the earthquake acceleration time history. Based on these results, the three iterations for the proposed dynamic analysis were performed using the following sections of the earthquake time history:

Iteration 1: From 10 to 30 seconds
Iteration 2: From 10 to 30 seconds
Iteration 3: From 10 to 70 seconds

Conclusions: (Information to follow on completion of computer anaysis in mid-February 1982).

(g) Instrumentation

Instrumentation will be installed within all parts of the dam to provide monitoring during construction as well as during operations. Instruments for measuring interval vertical and horizontal displacements, stresses and strains, and total of fluid pressures, as well as surface monuments and markers will be installed. The quantity and location will be decided during final design. Typical instrumentation is as follows:

- Piezometers

Piezometers are used to measure static pressure of fluid in the pore spaces of soil rockfill and in the rock foundation.

- Internal Vertical Movement Devices

. Cross-arm settlement devices as developed by the USBR.

. Various versions of the taut-wire devices have been developed to measure internal settlement.

. Hydraulic-settlement devices of various kinds.

- Internal Horizontal Movement Devices

. Taut-wire arrangements.
. Cross-arm devices.
. Inclinometers.
. Strain meters.

- Other Measuring Devices

. Stress meters.
. Surface monuments and alignment markers.
. Seismographic records and seismoscopes.
. Flow meters to record discharge from drainage and pressure relief system.

12.7 - Relict Channel Treatment

(a) Site Conditions

Earlier studies identified a buried channel running from the Susitna River gorge immediately upstream from the proposed damsite to Tsusena Creek, a distance of about 1.5 miles. Boring by the Corps of Engineers penetrated 454 feet of glacial deposits overlying bedrock which was encountered at elevation 1,775 feet, while the surface elevation of the lowest saddle is approximately 2,005 feet. Additional investigations during the current study further delineated the channel and full details are given in the Task 5 Geotechnical Report. The channel represents a potential source of leakage from the Watana reservoir. Along the buried channel thalweg, the highest bedrock surface is some 450 feet below reservoir level, while along the shortest leakage path between the reservoir and Tsusena Creek the highest rock surface is some 250 feet below reservoir level. The maximum hydraulic gradient along the bured channel from the edge of pool to Tsusena Creek, is approximately 9 percent, while between existing riverbed levels it decreases to about 6 percent. There are surface lakes within the channel area and while some drill holes encountered artesian water, others penetrated highly permeable zones resulting in complete loss of drilling fluid. Zones of permafrost have also been identified throughout the channel area.

Although the glacial history of the area is not clearly understood, a sequence of events has been postulated in the Task 5 Geotechnical Report, based primarily on the investigation of the Borrow Area D adjacent to the buried channel. The generalized surficial stratigraphic column is given in Table 12.5 and the gradation of the soils in the various units are given in Figures 12.55 through 12.18.

Of particular relevance to the buried channel problems are the alluvium at the base of the channel, encountered in one deep borehole between 292 feet and bedrock at 454 feet below ground, and the unconsolidated outwash, alluvial and fluvial deposits. The deep alluvium offers a potential leakage path, its high permeability being indicated by loss of drilling fluid, while the unconsolidated, primarily sandy deposits may be subject to liquefaction following saturation.

(b) Potential Problems

The major potential problems associated with the buried channel are leakage, both surface and subsurface flows; piping at downstream outlets to Tsusena Creek; the impact of permafrost and the long-term effects as heat from the reservoir thaws the ground through the channel area; and instability of soil slopes on saturation, thawing, or seismic loading leading to a breach of the rim of the reservoir.

(i) Surface Flows

During the study of alternative layouts for Watana, the maximum operating reservoir level was higher than the critical ground elevation of 2005 in the buried channel area. These layouts, therefore, incorporated a saddle dam about 40 feet high and 2,500 feet long across the critical section of the channel. The foundation conditions for such a saddle dam are not well defined at this time but because of the variable nature of the glacial deposits, the effects of permafrost and potential for liquifaction within the foundation were addressed. It was concluded, however, that in any event there was a strong possibility that settlement of such a dam could not be adequately controlled and there would be a real risk of transverse cracking occurring through the dam. With the reservoir level above ground surface, any such cracking could lead to surface flows and subsequent channeling through the unconsolidated deposits.

(ii) Subsurface Leakage

No field permeability tests have been conducted, but it is anticipated that the total subsurface leaking will be relatively small and economically insignificant. For example, if the average permeability of all material in the channel were 10^{-2} cm/sec, the total leakage flow would be less than 100 cubic feet per second. By inspection of the grading curves, the actual permeability is certainly less than 10^{-2} cm/sec, except possibly in the channel bed alluvium, and a more realistic leakage flow would be about 10 cubic feet per second. The capital value of this leakage is about $4 million. However, any leakage may be concentrated in the discharge zone in Tsusena Creek, and there is potential for piping which could lead to large-scale erosion cutting back to the high ground forming the rim of the reservoir.

(iii) Permafrost

Thawing of permafrost will result in higher seepage rates and possibly settlement of the surface as excess water drains from the thawed soils.

(iv) Liquefaction

Filling the reservoir will lead to the saturation of some of the glacial deposits within the buried channel area, including the upper slopes of the Susitna River valley, and produce the potential for liquefaction of these deposits under seismic loading. Under extreme circumstances, liquefaction could lead to mass movements of soils into the reservoir and breach of the reservoir rim in the area of the freeboard dike.

For this situation to occur, it would require a large, continuous deposit of loose, saturated, granular material with sufficient ground surface slope so that the soil above the liquefied zone would move under its own height. Although such a scenario is considered most unlikely, the investigations to date are not sufficiently detailed to preclude the possibility. In view of the potentially catastrophic failure that would result from a breach of the reservoir rim, further investigations must be carried out prior to construction to confirm the stratigraphy and provide adequate data to properly assess the need for and design of remedial treatment.

(c) Remedial Measures

Since the stability of the section of the buried channel forming the rim of the Watana reservoir is essential for the feasibility of the Watana development as outlined in this report, practical solutions to all possible scenarios, including extreme combinations of the problems outlined above, must be identified.

(i) Surface Flows

To eliminate the potential problems associated with settlement and breach of a saddle dam allowing surface flows through the buried channel area, the maximum operating level of the reservoir has been lowered to 2,185 feet leaving a width of at least 1,500 feet of "dry" ground at the saddle above this elevation. A freeboard dike with a crest elevation of 2010 is required to provide protection against extreme reservoir levels under probable maximum flood conditions. The shortest distance between the toe of the dike and the edge of the 2185-elevation reservoir pool is at least 450 feet, and

and under PMF flood, the water level will just reach the toe of the dike.

(ii) Subsurface Flows

Progressive piping and erosion in the area of discharge into the Tsusena Creek will be controlled by the placement of properly graded granular materials like a filter blanket over the zones of emergence. Field investigation will be carried out to define critical areas, and only such areas will be treated. Continuous monitoring of the outlet area will be necessary, since it may take many years for equilibrium with respect to permafrost to become established in the buried channel area.

If the permeability of the base alluvium is found to be excessive, grouting of the upstream inlet zone could be carried out to reduce the total leakage.

(iii) Permafrost

Thawing of permafrost will occur that may have an impact on subsurface flows and ground settlement. No specific remedial work is necessary; but flows, ground water elevation, and ground surface elevation in the buried channel area must be monitored and any necessary maintenance work carried out to maintain freeboard and control seepage discharge.

(iv) Liquefaction

To guarantee the integrity of the reservoir rim through the channel area requires that either:

- There is no potential for a liquefaction slide into the reservoir which could cut back and breach the rim, or

- If there is such potential, there is a sufficient volume of stable material at the critical section that even if the upstream materials were to slide into the reservoir, the failure zone could not cut back to the reservoir rim.

Any remedial treatment required will depend on the location and extent of critical zones and could range from stabilization by compaction (vibroflotation) or grouting techniques, either cement or chemical grouting, or in the limit, removal of material.

The stratigraphic column indicates that the two lower till deposits I and K have been overconsolidated by glaciation, and it is unlikely that these deposits could liquefy under any circumstances. The overlying Unit H is a medium fine sand with silt and is probably the most susceptible to liquefaction of all the materials sampled. This unit has been identified up to 40 feet thick in places, with the top of the layer estimated to be about 100 feet below ground surface at the deepest point, as shown on Plate 6.34 of the Task 5 Report. All materials above this unit are normally consolidated water/lain till, outwash, alluvium, and fluvial deposits which could include zones of critical materials.

There are insufficient data available to identify the full extent of such critical materials; hence, it is not possible to precisely define the remedial work necessary at this time. Available alternative methods include:

- Densification

  Layers within about 100 feet of the surface could be compacted by vibroflotation techniques to eliminate the risk of liquefaction and provide a stable zone.

- Stabilization

  Critical layers at any depth could be grouted, either with cement for fine gravels and coarse sands or by chemical grouting for fine sands and silts.

- Removal

  This could range from the replacement of critical material near the valley slopes with high-quality, processed material, which would stabilize the toe of a potential slide and so prevent the initiation of failure that might otherwise cut back and cause major failures, to the excavation blending and replacement of large volumes of material to provide a stable zone.

The ultimate treatment will be based on an engineering and cost analysis study of the appropriate alternatives during the design phase of the project when the site conditions have been more closely defined. However, to confirm the technical and overall financial feasibility of the project at this time, it is necessary to consider a solution to the worst conditions deemed possible.

On the basis of available data, such conditions are:

- That the alluvium Unit H encountered between elevations 2100 and 2140 in drill hole DR22 is a homogeneous loose, silty, fine sand.

- That it is of large areal extent and continuous from beyond the saddle out to the Susitna valley slopes.

- That is of such thickness that a failure plane could be contained fully within its boundaries.

With these conditions, liquefaction of the unit under seismic loading following saturation from the reservoir could result in the overlying material sliding on the liquefied zone into the reservoir.

Catastrophic failure would develop if the back scarp of the failure surface through the overlying materials broke ground surface on the downstream side of the saddle below reservoir water level.

The most positive solution to such a situation would be the replacement of the critical zone with material that would not liquefy. This would involve, in effect, the rearrangement of the in-place materials to create an underground dam section founded on the dense till layer beneath the critical alluvium. Such an operation would involve the excavation of a trench up to 135 feet deep with a surface width up to 1,000 feet. Selected materials would be compacted to form a central zone, with 2 horizontal to 1 vertical slopes. Surplus and unsuitable materials would be placed on both sides of this central "dam" to complete backfilling to ground surface. The central zone would be designed to remain stable in the event that all material upstream did slide into the reservoir. Preliminary estimates indicate that such a structure would need to be 5,000 feet long, with a total cut volume of about 13 million cubic yards, of which 4-1/2 million cubic yards could be used in the compacted center zone. The cost of such work is estimated to be about $100 million. The need of such expenditure is considered to be most likely and is deemed to be covered by the overall project contingency sum.

(d) Further Investigations

Additional site investigations are necessary in the relict channel area to more closely define the following:

- Confirm and/or refine the stratigraphy throughout the area.

- Thickness, extent, density, continuity, and permeability of the alluvium identified in DR22 immediately above bedrock. The investigations should include pumping tests and dye injection tests to check the continuity of this unit along the buried channel, since this is deemed to have the greatest potential for leakage.

- Density of the lowest till layers I and J which have been subjected to overconsolidation by glaciation to confirm that they would not liquefy under earthquake loading.

- Density, gradation, extent, and continuity of the sandy silt alluvium, Unit H.

- Extent of any other units which may be subject to liquefaction.

- Conditions in the outlet area of the relict channel into Tsusena Creek.

- Ground water regime throughout the channel area with particular reference to the source of artesian or confined aquifers and the drainage outlets from such aquifers.

(e) Construction Methods

For the purpose of this feasibility study, it has been assumed that treatment of the relict channel will involve the following:

- Construction of the freeboard dike at the crest of the saddle involving stripping of organic material and construction of the dike with impervious core and gravel or rock-fill shells; and

- Stripping, grading, and placement of a filter blanket over the outlet area of the channel in Tsusena Creek. This blanket is estimated to be 7 feet thick and cover an area of 460 acres.

Allowance for the cost of these items is included in the estimates. However, it is considered likely that the actual area of downstream blanket required will be less than allowed, but that other remedial work in the saddle area will probably be necessary.

The allowance for the downstream blanket is therefore considered a reasonable estimate to cover all work required in the channel area excluding the extreme situation involving major excavation as discussed above.

12.8 - Primary Outlet Facilities

The primary function of the outlet facilities will provide capability to discharge floods with recurrence frequencies of up to 1:50 years after they have been routed through the Watana reservoir. Downstream erosion will be minimal and the dissolved nitrogen content in the discharges will be restricted as much as possible to avoid harmful effects on the downstream fish population. A secondary function of the facilities will be to enable the rapid drawdown of the reservoir by up to 150 feet during an extreme emergency situation.

The structures will be located on the right abutment, as shown on Plate 48, and will consist of an intake structure, pressure tunnel, and an energy dissipation and control structure housing six fixed-cone valves which will discharge into the river 150 feet below.

(a) Approach Channel and Intake

The approach channel to the outlet facilities will be shared with the power intake. The channel will be 400 feet wide and excavated to a depth of approximately 150 feet in the bedrock with an invert elevation of 2007. The intake structure will be founded deep in the rock at the end of the channel. The single inlet passage will have an invert elevation of 2003. It will be divided upstream by a central concrete pier which will support steel trashracks located on the face of the structure, spanning the openings to the water passage. The racks will screen out submerged debris which could damage the downstream valves. The racks will remain in place with no provision for their withdrawal, which would be expensive and is considered unnecessary because of the infrequent use of the outlet. If there ever should be a blockage, this would have to be cleared by divers. Downstream of the racks, located between the pier and each of the sidewalls, will be two fixed wheel gates operated by a mechanical hoist mounted (at the surface) above the deck of the structure. The purpose of the fixed wheel gates will not be to control flows through the outlet, but to close off the downstream tunnel to allow dewatering for maintenance of the tunnel or gates within the discharge structure. Stoplog gates will be provided just upstream of the gates to allow insertion of stoplogs for dewatering of the structure and access to the gate guides for maintenance.

(b) Intake Gates and Trashracks

The gates will be of the fixed wheel vertical lift type with downstream skinplate and seals. The nominal gate size will be 16 feet wide by 32 feet high. Each gate will be operated by a single drum wire rope hoist mounted in an enclosed tower structure at the top of the intake. The height of the tower structure will permit raising the gates clear of the intake concrete for inspection and maintenance.

The gates will be capable of being lowered either from a remote control room or locally from the hoist area. Gate raising will be from the hoist area only.

The trashracks will have a bar spacing of about 7 inches, and will be designed for a maximum differential head of about 40 feet. The maximum net velocity through the racks will be about 6 ft/s. Provision will be made for monitoring the head loss across the trashracks.

(c) Shaft and Tunnel

Discharges will be conveyed from the upstream gate structure by a concrete-lined shaft and tunnel terminating in a steel liner and manifold. The manifold will branch into six 78-inch diameter steel-lined tunnels which will run through the main spillway flip bucket structure to the valves mounted on the downstream face.

The water passages will be 28 feet in diameter up to the steel manifold. The upstream concrete-lined portion will run a short distance horizontally from the back of the intake structure before dipping at an angle of 55° to a lower level tunnel of similar cross section. This angle of 55° is considered the flattest slope at which the tunnel can be "self-mucking" during construction and is cost effective in the resultant ratio of shaft to tunnel length. The lower tunnel will run at a gradient of 1:10 to the point where the overlying rock is insufficient to withstand the large hydrostatic pressure which will occur within the tunnel. Downstream of this point the pressure will be transferred throughout the mass concrete and resisted jointly by the surrounding rock and the steel liner. The steel liner will be 26 feet in diameter and surrounded by mass concrete filling the space between the liner and the surrounding rock. The area of the outside face of the liner and the concrete will be grouted to fill all voids and reduce non-uniform external ground water pressure build up from the ground water.

Upstream from the discharge structure the liner will terminate in a steel manifold with six parallel 8 foot diameter steel-lined branches. These will continue through the back face of the discharge structure, and terminate in discharge valves mounted at the downstream end of the structure.

(d) Discharge Structure

The concrete discharge structure is shown on Plate 52. It will form the flip bucket for the main spillway and will house the fixed cone valves and individual upstream ring follower guard gates. It will be founded on sound rock high above the river The valves will be set with a centerline elevation of 1,560 feet and will discharge into the river approximately 85 feet below. Openings for the valves will be formed in the concrete and the valves will be recessed within these openings sufficiently to allow enclosure for ease of maintenance and heating of the moveable valve sleeves. An access gallery upstream from the valves will run the length of the discharge structure, traversing the steel liners upstream of the valves, and will terminate in the access tunnel and access road on either side of the structure. Housing ring follower guard gates will be located upstream from the gallery gate chambers. These gates will operate on the steel liners and will serve to isolate the discharge valves. A common monorail hoist will be located above each valve and gate assembly to provide for their removal and transportation to the access gallery. From the access gallery they can be maneuvered on a low trolley to the service area adjacent to the end of the gallery.

(e) Fixed Cone Discharge Valves

Eight 78-inch diameter fixed cone discharge valves will be installed at the downstream end of the outlet manifold, generally as shown on Plate 52. The valves were selected to be within current experience, considering the valve size and operating head (see Figure 12.20). The fixed cone valves are similar to Howell Bunger valves except that the cone support valves extend further upstream and are more streamlined. The valves have a slightly higher discharge coefficient than Howell Bunger valves and are less prone to vibration.

Electric jacket heaters will be installed around the cylindrical sleeves of the valves which extend outside of the valve room, and since the valves will be located within a heated valve room, they will be capable of year-round operation. Normally, when the valves are closed, the upstream ring follower gate will also be closed, so that freezing of leakage at the valve seat should not be a serious concern.

The valves will be operated either by two hydraulic cylinder operators or by a screw stem hoist. For preliminary design purposes, hydraulic operators have been assumed. The valves may be operated either locally or remotely.

At the time of valve design, careful consideration must be given to prevent vibration. Considerable research will be carried out concerning experience and design of existing installations, and model tests will be necessary to help ensure satisfactory valve operation. Adequate design of the fixed vanes will be of prime importance.

In sizing the valves for the preliminary design, it has been assumed that the valve gate opening will be restricted to 80 percent full stroke because of vibration considerations.

Careful consideration must also be given to machining and surface finish of the valves, in order to prevent cavitation and erosion of the valve seals resulting from the extremely high water velocities.

(f) Ring Follower Gates

A ring follower gate will be installed upstream of each valve and will be used:

- To permit inspection and maintenance of the fixed cone valves;
- To relieve the hydrostatic pressure from the valves when they are in the closed position; and
- To close against flowing water in the event of malfunction or failure of the valves.

The ring follower gates will have a nominal diameter of 90 inches and will be of welded or cast steel construction. The gates will be designed to withstand a total static head of about 630 feet. Existing large diameter high head ring follower gates are summarized in Table 12.6.

The ring follower gates will be designed to be lowered under flowing water conditions with maximum head, but they will only be raised under balanced head conditions. Valved bypass piping will be used to equalize the pressure on both sides of the ring follower gate before raising. The gates will be operated by hydraulic cylinders with a nominal operating pressure of 2000 psi. Either local or remote operation of the ring follower gates will be possible.

A grease system will be installed in each gate for injection of grease between the gate leaf and the gate body seats to reduce frictional forces when the gates are operated.

(g) Discharge Area

Immediately downstream of the discharge structure, the rock will be cut at a slope of 2H:3V to a lower elevation of 1,510 feet. This face will be heavily reinforced by rock bolts and protected by a concrete slab anchored to the face. The lower level will consist of unlined rock extending to the river. Much of the lower trajectory of the discharge jets will impinge on this area in the form of a very heavy spray, while much of the upper part of the jets will carry as far as the river. Because of the high degree of dispersion of the discharges and the infrequency of operation of the valves, it is not anticipated that erosion will be a problem requiring other than occasional minor maintenance.

12.9 - Main Spillway

(a) General

The main spillway will provide discharge capability for flows exceeding the capacity of the outlet facilities, passing the excess of floods with a corresponding return period of less than 1:50 years. The combined total capacity of the main spillway and outlet facilities will be sufficient to pass routed floods with recurrence frequencies of up to 1 in 10,000 years.

The spillway, shown on Plate 49, is located on the right abutment and consists of an approach channel, a gated ogee control structure, a concrete-lined chute, and a flip bucket.

(b) Approach Channel and Control Structure

The approach channel is excavated to a depth of approximately 100 feet into rock. It is adjacent to the power facilities approach channel, and in order to minimize its length, it partly offtakes from this channel, intersecting it at elevation 2125 feet.

The control structure is a massive concrete structure set at the end of the approach channel. Flows are controlled by three 42 feet high by 36 feet wide vertical lift gates. As shown on Plate 50, each gate is contained within a separate unit consisting of an ogee overflow weir, piers, and an integral roadway deck. The units are of amonolithic box type structure with split pier construction. The box configuration will give rigidity during seismic shaking, and the split piers will allow for some relative motion and stress relief during earthquake motion in order to minimize the possibility of the gates jamming in the closed position.

Model tests will be necessary during the final design stage to determine final geometry and dimensions such as pier noses, crest shape, and pier length.

The structure will be located adjacent to the right dam abutment in line with the dam crest. The main access route from the Denali Highway will pass across the spillway deck and along the crest.

The main dam grout curtain and drainage system will pass beneath the structure. Access to the grouting tunnels will be via a shaft within the structure and a gallery running through the ogee weir.

(c) Spillway Gates and Stoplogs

The three spillway gates will be of the fixed wheel vertical lift type operated by double drum wire rope hoists located in an enclosed bridge structure. The gate size has been selected as 36 feet wide by 40 feet high, including a 3 foot freeboard allowance above maximum normal reservoir level. The gates will have upstream skinplates and seals will be totally enclosed to permit gate heating in the event that winter operation is necessary. Provision will also be made for heating the gate guides.

The height of the tower and bridge structure will be selected to permit raising of the gates above the top of the spillway pier for gate inspection and maintenance.

An emergency gasoline engine will be provided to enable the gates to be raised in the event of loss of power to the spillway gate hoist motors.

A set of stoplog guides will be installed upstream of each of the three spillway gates. One set of stoplogs will be provided to permit raising the gates with maximum reservoir levels without discharging water over the spillway.

The stoplogs will have downstream skinplate and seals and will be arranged in sections suitable for handling by a mobile crane with a follower.

(d) Spillway Chute

The control structure will discharge down an inclined chute that tapers slightly to a width of 80 feet, which is maintained over the remainder of its length.

The chute section will be rectangular in cross section, excavated in rock, and lined with concrete which will be anchored to the rock. An underdrainage system will be constructed beneath the slabs to relieve uplift pressures and will be formed by drilling holes from a centrally located gallery, in the rock, to intersect box drains located beneath the chute floor slab. Provision will be made at two locations down the chute to aerate flows and prevent cavitation of the concrete floor. Aeration will be attained by means of an inclined step into the flow. Air will be drawn of from a transverse lower gallery via ducts which exit at the downstream vertical face of the step. Convergence of the chute walls at the upper end of the spillway will be gradual to minimize any shock wave development. Model tests will determine the maximum allowable convergence of the walls to assure both economy and satisfactory hydraulic performance. Model tests will also determine aeration requirements (number and size of aeration ducts) and the shape of the chute floor.

(e) Flip Bucket

The function of the flip bucket will be to direct spillway flows clear of the spillway and well downstream into the river below. The jet issuing from the bucket will be partly dispersed during its passage through the air with a corresponding loss of energy. The remainder of the energy will be dissipated on impact with the plunge pool.

The mass concrete block, as described in Section 12.8, will house the gates and discharge valves for the outlet facilities, and also will form the flip bucket for the main spillway. The elevation of the structure will be controlled by the elevation and head limitations of the discharge valves. Final geometry of the bucket will be determined by model studies, as well as dynamic pressures on the floor and walls of the structure. Although the structure shown on Plate 52 shows a simple, cylindrical type of bucket, it it is foreseen that a more effective, dispersive type bucket will be developed during model tests.

Such a bucket was developed for the Portage Mountain Project in British Columbia, where flows up to ____ cfs under a static head of ____ feet are discharged at a 90° angle into the river below. In order to prevent erosion of the opposite river bank, the jet is spread by means of a disk-shaped bucket so that the area of impingement in the river is spread in a narrow line parallel to the bank and normal to the chute centerline.

12.10 - Emergency Spillway

The emergency spillway will be located on the right side of the river beyond the main spillway and the power intake structure (see Plate 53). The spillway will consist of a long straight chute cut in the rock and leading in the direction of Tsusena Creek. An erodible fuse plug, consisting of impervious and fine gravel materials, will be constructed at the upstream end; it will be designed to wash away when overtopped by the reservoir, thus releasing floods of up to 160,000 cfs in excess of the combined main spillway and outlet capacities and preventing overtopping of the main dam.

(a) Fuse Plug and Approach Channel

The approach channel to the fuse plug will be excavated in rock and will have a width of 310 feet and invert elevation of 2170. The main access road to the dam and powerhouse will cross the channel by means of a bridge. The fuse plug will close the approach channel, and will have a maximum height of 31.5 feet with a crest elevation of 2201.5 feet. The plug will have an impervious core up to 10 feet wide, steeply inclined in an upstream direction, with fine filter zones upstream and downstream. It will be supported on a downstream erodible shell of crushed stone or gravel up to 1.5 inches thick. The crest of the plug will be 10 feet wide and will be traversed by a 1.5 foot deep pilot channel. The zoning of the plug will be similar to that designed for the New Melones Lake project in California. For this project a half scale version of the plug was constructed and tested to destruction. The plug failed completely in less than an hour and the water level rose no more than 6 inches above the invert of the pilot channel. The principle of the plug is based on erosion progressing rapidly downward and laterally from the pilot channel as soon as it is overtopped.

The channel section at the fuse plug is considered as a broad crested weir with a coefficient of discharges of 2.7. A gated control structure was considered as an alternative to the fuse plug, but this would give higher construction and maintenance costs and would not provide the automatic breaking and discharge of the plug.

(b) Discharge Channel

The rock channel downstream of the fuse plug will narrow to 200 feet and continue in straight line over a distance of 5,000 feet at gradients of 1.5 percent and 5 percent in the direction of Tsusena Creek. The channel will discharge in a buried valley on the downstream side of the main relict channel from which flows will run down into the creek. It is estimated that flows down the channel will continue for a period of ___ days under probable maximum flood conditions. Some erosion in the channel will be acceptable, but the integrity of the main dam will be maintained. The reservoir will be drawn down to Elevation 2170, and reconstruction of the fuse plug will be required prior to refilling of the reservoir.

12.11 - Intake

(a) General

The intake structure at Watana will be required to fulfill the following functions:

- To provide independent power flow to each of six Francis turbines, up to a maximum flow of 3,800 cfs per unit, for any reservoir level from EL 2000 (maximum extreme flood level) to the minimum drawdown level (Selection of drawdown level is reviewed below);

- To provide an upstream control gate on each penstock to permit dewatering of the penstock and turbine water passages for routine inspection and maintenance; and

- To control the temperature of water discharged from the reservoir within acceptable limits to mitigate the environmental impacts of the Susitna development on downstream fisheries and vegetation.

(b) Environmental Constraints

The seasonal operation of the Watana reservoir will have two major impacts on downstream flows:

- In summer, the temperature of downstream releases will be cooler than the normal river regime; and

- In winter, the temperatures will be warmer than the normal river regime.

Cooler water in the summer months could have a significant impact on downstream fisheries, particularly in July and August when salmon are moving into the sloughs downstream from Devil Canyon to spawn. Warmer water in winter will affect the formation of ice, resulting in extensive open water downstream from the reservoirs.

Temperature simulation using a Corps of Engineers Hydraulic Engineering Center (HEC) program was used to model the downstream effects of reservoir operation using a variety of different power intake designs at Watana and Devil Canyon. These studies have indicated that downstream temperatures will remain constant in winter at 39°F regardless of the type of power intake design used. However, the downstream river temperatures in the summer months can be significantly improved by power intake design at Watana which months can be significantly improved by power intake design at Watana which would permit water to be drawn off to the reservoir surface at all times. The power intake design at Devil Canyon will be less significant because the maximum drawdown is only 50 feet.

The selected power intake design at Watana will permit water to be drawn from the reservoir at four distinct levels through the anticipated range of drawdown to mitigate the environmental impacts on downstream river temperatures (see Volume 2).

Details of the reservoir temperature modeling are presented in Appendix B4.

(c) Drawdown

The maximum drawdown at the power intake controls the live storage of the reservoir (the volume available for seasonal and over year regulation of the river flows). With no drawdown capability, the dependable (firm) energy from Watana would be controlled by the dependable flow of record to the required reliability criterion; this has been defined as the lowest flow in the second driest year of record, with a recurrence of about 1 in 70 years.

As the drawdown is increased, the firm flow from the reservoir will increase. The firm energy will also increase for drawdown up to about 140 feet. If drawdown is increased beyond 140 feet, however, the firm flow will increase but the firm energy will start to reduce, since dependable energy is governed by a combination of flow and head.

Average energy available from the reservoir shows a very slight decrease with increasing drawdown because of the imposed constraint in the computer simulation that the reservoir should be full at the end of the 32 years of recorded flows. However, the usable energy will increase with drawdown because the storage available will permit excess energy in summer to be stored for use in the winter.

Costs of the intake structure and approach channel will increase significantly with the depth of drawdown, while the cost of penstocks will be reduced. A detailed study has shown that the extra benefit of increased firm energy from Watana will be in excess of the required incremental cost of the intake, approach channel, and penstocks for any drawdown up to 140 feet. Therefore, the maximum drawdown selected for preliminary design was 140 feet.

(d) Design

The power intake will be a free-standing concrete structure located in rock excavation at the upstream end of the approach channel. Access to the structure will be the same as access to the intake for the outlet works, since the two structures have a common approach channel.

In order to draw water close to the reservoir surface over a drawdown range of 140 feet, four openings will be provided in the upstream concrete wall of the structure for each of the six independent power intakes. The upper opening will always be open, but the lower three openings can be closed off by sliding steel shutters operated in a common guide. All openings will be protected by upstream trashracks. A heated ice bulkhead will be provided for winter operation.

An intake control gate will be provided on each penstock. A single upstream bulkhead gate will be provided for routine maintenance on the six intake control gates. In an emergency, stoplogs can be installed in the upstream wall of the power intake for work on the trashracks or shutter guides.

The width of the intake will be controlled by the minimum spacing of penstock tunnel excavations, taken as 2.5 times the excavation diameter.

The upper level of the concrete structure will be set at EL 2200, corresponding to the maximum anticipated flood level. The minimum structure level will be governed by the physical dimension of the penstock tunnel excavation and the vortex criterion for flow into the penstock from the minimum reservoir level EL 2045. The foundation of the structure will be about 150 feet below existing ground level and will be expected to be generally in sound rock.

Mehanical equipment will be housed in a steel-frame building on the upper level of the concrete structure. The general arrangement of the power intake is shown on Plate ____.

(e) Approach Channel

The width of the approach channel will be governed by the combined width of the power intake and the intake to the outlet facilities, which in turn will be governed by the minimum spacing of the penstock and outlet tunnels. The overall width of the channel will be about 350 feet. Excavated slopes in sound rock will be generally 1H:4V.

The maximum flow in the intake approach channel will occur when six machines are operating and the outlet facilities are discharging at maximum design capacity, with the reservoir drawdown to EL 2045. Under these conditions the maximum velocity of flow in the approach channel will be 3.5 ft/s, which will not cause any erosion problems.

Higher flows will be possible where the intake approach channel intersects the approach channel to the main spillway and the approach channel to the emergency spillway. The maximum velocity of flow will be about 25 ft/s, and excavated slopes in this vicinity may need increased support at localized areas of sheared or fractured rock.

Excavations in overburden will generally be trimmed at 2H:1V; rip-rap protection will be required in the areas where high-flow velocities are anticipated.

(f) Geotechnical Considerations

The excavation will be over 200 feet deep in rock in the northwest corner, with a total excavation depth of 240 feet. The southern end of the structure will be located in the 80 feet with shear and fracture zone. The excavation depth at the north end of the structure will be 120 feet.

With sufficient rock support, mainly from rock bolting, the rock slopes can be cut nearly vertical, with the possible exception of the southern end, where the excavation will intersect the fracture and shear zone. If it proves impracticable to support this face nearly vertically, it will have to be trimmed back to a stable slope. The intake structure would then be partially free-standing. The spillway tunnel portal will also be located in this zone of fractured rock and will require substantial rock support installed in the portal face. Since the intake structure will, when complete, support this rock face, the required support will be temporary.

The foundation will be in sound rock, but the shear and fracture zones at the southern end may require consolidation grouting. Minor shears and fractures exposed in the remainder of the foundation area may require local grouting and/or dental concrete.

(g) Mechanical Arrangement

(i) Ice Bulkhead

A heated ice bulkhead will be installed in guides immediately upstream of the trashracks for each of the six power intakes. The ice bulkhead will be operated by a movable hoist with a double point lift and will be automatically raised and lowered so that it will always be at reservoir level. The ice bulkhead will serve to:

- Minimize ice accumulation in the trashrack and intake shutter area; and

- Prevent thermal ice-loading on the trashracks.

The bulkhead will incorporate fixed wheels which will serve to transmit thermal ice loads to the embedded guides. The bulkhead will be totally enclosed and heated. The power supply cable to the bulkhead will be located on an electric, motor-operated, take-up reel which will operate in conjunction with the bulkhead hoist. The bulkhead will be handled by a movable hoist to facilitate removal of the bulkhead for trashrack cleaning.

In the unlikely event that it is desired to dewater the complete intake for inspection of the shutter guides, stoplogs can be purchased and installed in the ice bulkhead guides. The height of the stoplogs will depend on the reservoir level at the time of inspection.

(ii) Trashracks

Each of the six power intakes will have four sets of trashracks, one set in front of four intake openings. Each set of trashracks will be in two sections to facilitate handling by the intake service crane with a follower. Each set of trashracks will cover an opening 30 feet wide by 24 feet high. The trashracks will have a bar spacing of about 6 inches and will be designed for a maximum differential head of about 25 feet.

(iii) Intake Shutters

Each of the six power intakes will have three intake shutters which will serve to prevent flow through the intake openings behind which the shutters will be installed. As the reservoir level drops, the sliding shutters will be removed as necessary using the intake service crane.

Each of the shutters will be designed for a differential head of about 15 feet. Suitable alarms and generating unit shutdown devices will be installed to activate when the differential head is about 1/2 this value. In addition, at least one of the shutters at each power intake will incorporate a flap gate which, with 15 feet differential head across the shutter, will allow maximum turbine flow through the flap. This will prevent failure of the shutters in the event of accidental blocking of all intake openings below water level.

The shutter guides will be heated to facilitate removal in subfreezing weather. In addition, a bubbler system will be provided in the intake behind the shutters to keep its water surface free of ice in this area. The bubbler nozzles will be located at several levels to permit bubbler system operation as the reservoir is drawndown.

(iv) Intake Service Crane

A single, overhead, traveling-bridge type intake service crane will be provided in the intake service buildings. The crane will be used:

- For servicing the ice bulkhead and ice bulkhead hoist;
- For handling and cleaning the trashracks;
- For handling the water intake shutters;
- For handling the intake bulkhead gates; and
- For servicing the intake gate and hoist.

The overhead crane will have a double point lift and will have followers for handling the trashrack shutters and bulkhead gates. The crane will be radio-controlled with a pendant or cab control for backup. A grappling hoist will be provided for cleaning debris from the rack area.

(v) Intake Bulkhead Gates

One set of intake bulkheads, consisting of two gate sections, will be provided for closing any one of the six intake openings upstream from the intake gates. The gates will be used to permit inspection and maintenance of the intake gate and intake gate guides. The gates will be raised and lowered under balanced water conditions only. To balance water pressure in order to raise the intake bulkhead gate, the space between the gate and the downstream control gate will be flooded by a follower-operated bypass valve on the top gate section; an air valve will be provided in the top of the gate. The gates will have a downstream skinplate and will seal on the downstream side. The gate will be designed to withstand full differential pressure.

(vi) Intake Gates

The six intake gates, one for each intake, will each be provided to close a clear opening 17.3 feet wide by 16.7 feet high. It is proposed that the gates will be of the vertical fixed wheel lift type with an upstream skinplate, and they will seal on the upstream side.

Each gate will be operated by a hydraulic cylinder type hoist, mounted below the 2,200-foot deck level. The length of a cylinder will allow withdrawal of the gate from the water flow. The cylinder and gate will be connected by a series of hooked links. The intake service will be used to raise the gate, complete with links and cylinder, by dogging and removing each link in turn until the gate is raised above deck level for maintenance. The gates will normally be closed under balanced flow conditions to permit dewatering of the penstock and turbine water passages for inspection and maintenance of the turbines. The gates will also be designed to close in an emergency with full turbine flow conditions in the event of loss of control of the turbine.

The hoist pumping unit will have an operating pressure of 1,000 to 2,000 psi. The hoist will be designed to allow gate closure without any ac power supply.

A heated air vent will be provided at the intake deck to satisfy air demand requirements when the gate is closed with flowing water.

12.12 - Penstocks

The general arrangement of the penstocks is shown on Plates 54 and 55.

The maximum design static head on each penstock is from normal full reservoir elevation (EL 2185) to centerline distributor level (EL 1422). An allowance of 35 percent has been made for pressure rise in the penstock because of hydraulic transients. The maximum design head is therefore 1,030 feet. Maximum extreme head (corresponding to maximum reservoir flood level) is 1,050 feet.

(a) Steel Liner

It has been assumed that the rock adjacent to the powerhouse cavern will be incapable of long-term restraint against the forces transmitted from penstock hydraulic pressures. Consequently, a steel liner will be required which will wholly resist the maximum design head, without support from the surround rock. This section of steel liner will extend 50 feet from the powerhouse. Beyond this section the steel liner will be extended a further 150 feet, and allowance in the design will be made for partial rock support to mitigate the maximum design stress. For preliminary design purposes it is assumed that not more than 50 percent of the maximum design head will be taken by the rock support over this transition length.

The steel liner will be surrounded by a concrete infill, with a minimum thickness of 24 inches. The optimum internal diameter of the steel lining will be 15 feet based on the minimum total cost of construction and the capitalized value of anticipated annual energy losses. A tapering transition will be provided to increase the internal diameter of the steel liner to 17 feet at the junction between the steel liner and the concrete liner.

(b) Concrete Lining

The penstocks will be fully lined with concrete from the intake to the steel lined section, the thickness of lining varying with the design head. The optimum internal diameter of the concrete lined penstock will be 17 feet, based on the minimum total cost of construction and the capitalized value of anticipated annual energy losses, and assuming an average concrete lining thickness of 18 inches. The minimum lining thickness will be 12 inches.

(c) Grouting and Pressure Relief

A comprehensive pressure relief system will be required to protect the underground caverns against seepage from the high pressure penstock. The system will comprise small diameter boreholes set out in patterns and curtains to intercept the jointing in the rock.

Grouting around the penstocks will be provided to:

- Seal and fill any voids between the concrete lining and the steel liner, which may be left after the concrete placing and curing; and

- Fill joints or fractures in the rock surrounding the penstocks to reduce flow into the pressure relief system and to consolidate the rock.

12.13 - Powerhouse

(a) General

The underground powerhouse complex will be constructed underground in the right abutment. This will require the excavation in rock of three major caverns, the powerhouse, transformer gallery, and surge chambers with interconnecting rock tunnels for the draft tubes and isolated phase bus ducts.

Unlined rock tunnels will be required for vehicular access to the three main rock caverns and the penstock construction adit. Vertical shafts will be required for personnel access to the underground powerhouse, for cable ducts from the transformer gallery, for surge chamber venting and for the heating and ventilation system.

The general layout of the powerhouse complex is shown in plan and section in Plates 54 and 55, and in isometric projection in Plate 56. The transformer gallery will be located on the upstream side of the powerhouse cavern; the surge chamber will be located on the downstream side. Clear dimensions between major rock excavations have been set at 1.5 times the main span of the larger excavation. This criterion controls not only the minimum distance between caverns, but also the spacing between transformer gallery and penstock, between bus shaft and penstock, and the minimum spacing of penstock and tailrace tunnels.

The draft tube gate gallery and crane will be located in the surge chamber cavern, above the maximum anticipated surge level. Provision will also be made in the surge chamber for tailrace tunnel intake stoplogs, which will be handled by the draft tube crane.

(b) Layout Consideration

The location of the powerhouse was selected from consideration of the following data:

- Plots of the known major faults and shear zones on the right abutment;

- Estimated cost of approach channel excavation, intake structure, penstocks, and tailrace; and

- An assumed angle of 55° to the horizontal for the inclined section of penstock.

Preliminary cost estimates indicate that the intake structure and approach channel excavation are the most significant items in the overall arrangement of the power facilities; the underground powerhouse costs are dependent only on installed capacity. The optimum arrangement has therefore been determined by adjusting the position of the intake to give the least cost for intake, penstocks, and tailrace. Since the costs of tunneling are small compared to the intake costs, the intake will be sited as far upstream as possible, consistent with the required minimum drawdown level, and a reasonable length of access tunnels.

The underground transformer gallery will be located on the upstream side of the powerhouse. This arrangement gives the minimum possible distance between the turbines and the surge chamber, for maximum protection of the draft tubes under transient load conditions. The transformer gallery and the powerhouse cavern will be protected against high pressure seepage from the penstocks by a 200 foot long steel-lined section and an extensive pressure relief system (see Section 12.12).

(c) Access Tunnels and Shafts

Vehicular access to the underground facilities at Watana will be provided by a single unlined rock tunnel from the right bank area adjacent to the diversion tunnel portal. The access tunnel will cross over the diversion tunnels and then descend at a uniform gradient to the south end of the powerhouse cavern at generator floor level, at EL 1463. Separate branch tunnels from the main tunnel will give access to the transformer gallery at EL 1507, the penstock construction adit at EL 1420, and the draft tube gate gallery at EL 1500. The maximum gradients will be 6.1 percent on the construction access tunnel, and 6.9 percent on the permanent access tunnels.

The common access tunnel will be sized to provide passing clearance for the construction plant used during penstock construction. The size of articulated trailer required to deliver heavy items of machinery such as the turbine runner, turbine spiral case, and generator rotor, will be less critical with respect to tunnel size, but will dictate the minimum radius of vertical and horizontal curves. For preliminary design, the cross section of the access tunnel has a modified horseshoe shape, 35 feet wide by 28 feet high. The access tunnel branch to the surge chamber and draft tube gallery will have a reduced section, consistent with the anticipated size of vehicle and loading required.

The main access shaft will be at the north end of the powerhouse cavern, providing personnel access from the surface control building by elevator. Access tunnels will be provided from this shaft for pedestrian access to the transformer gallery and the draft tube gate gallery. Elevator access will also be provided to the fire protection head tank, located about 250 feet above powerhouse level.

(d) Powerhouse Cavern

The main powerhouse cavern is designed to accommodate six vertical shaft Francis turbines, in line, with direct coupling to coverhung generators. Each unit is designed to generate 170 MW at a rated head of 680 feet.

The vertical dimension of the powerhouse cavern is determined by the physical size of turbine and generator, the crane height required for routine maintenance, and the design dimensions of the turbine draft tube. The length of the cavern will allow for a unit spacing of 60 feet, with a 110-foot long service bay at the south end for routine maintenance and for construction erection. The width of the cavern allows for the physical size of the generator plus galleries for piping and air-conditioning, electrical cables, isolated phase bus ducts, and generator circuit breakers. Continuous drainage galleries will be provided to a low level sump.

Vehicular access will be by tunnel to the generator floor at the south end of the cavern; pedestrian access will be by elevator from the surface control building to the north end of the cavern. Multiple stairway access points will be available from the main floor to each gallery level. Access to the transformer gallery from the powerhouse will be by tunnel from the main access shaft, or by stairway through each of the isolated phase bus shafts. A service elevator will be provided from the maintenance area on the main floor level to the machine shop and stores area on the turbine floor level.

Hatches will be provided through all main floors for installation and maintenance of heavy equipment using the overhead traveling cranes.

(e) Transformer Gallery

The transformers will be located underground in a separate gallery, 130 feet upstream from the main powerhouse cavern, with three connecting tunnels for the isolated phase bus. There will be nine single-phase transformers rated at 15/345 kV, 122 MVA, installed in groups of three one group for each pair of turbines. Generator circuit breakers will be required, and will be installed in the powerhouse on the lower generator floor level.

High voltage cables will be taken to the surface by two cable shafts, each with an integral diameter of 7.5 feet. Provision has been made for installation of an inspection hoist in each shaft. A spare transformer will be located in the transformer gallery, and a spare HV circuit will also be provided for improved reliability. The station service auxiliary transformers (2 MVA) and the camp services auxiliary transformer (7.5/10 MVA) will be located in the bus tunnels. Generator excitation transformers will be located in the powerhouse on the main floor.

Vehicle access to the transformer gallery will be the main powerhouse access tunnel at the south end. Pedestrian access will be from the main access shaft or through each of the three isolated phase bus tunnels.

(f) Surge Chamber

A surge chamber will be provided 120 feet downstream from the powerhouse cavern to control pressure rise in the turbine draft tubes and tailrace tunnels under transient load conditions, and to provide storage of water for the machine start-up sequence. The chamber will be common to all six draft tubes, and under normal operation will discharge equally to the two tailrace tunnels.

The draft tube gates gallery and crane will be located in the same cavern, above the maximum anticipated surge level. The draft tube gate crane has also been designed to allow installation of tailrace tunnel intake stoplogs for emergency closure of either tailrace tunnel.

The chamber will generally be an unlined rock excavation, with localized rock support as necessary for stability of the roof arch and walls. The gate guides for the draft tube gates and tailrace stoplogs will be of reinforced concrete, anchored to the rock by rockbolts.

Access to the draft tube gate gallery will be by an adit from the main access tunnel; the tunnel will be widened locally for storage of draft tube bulkhead gates and stoplogs.

12.14 - Reservoir

The Watana reservoir, at normal operating level of 2,185 feet, will be approximately 48 miles long with a maximum width in the order of 5 miles. The total water surface area at normal operating level is 38,000 acres. Just upstream from the dam, the maximum water depth will be approximately 680 feet. The minimum reservoir level will be 2,045 feet during normal operation, resulting in a maximum drawdown of 140 feet. The reservoir will have a total capacity of 9,515,000 acre-feet of which 4,210,000 acre-feet will be live storage.

Prior to reservoir filling, the area below Elevation 2190, five feet above maximum operating level, will be cleared of all trees and brush. A field reconnaissance of the proposed reservoir area was undertaken as part of these studies. This work included examination of aerial photographs and maps, an aerial overflight of the reservoir and collection or recent (1980 field season) forest inventory data from the U.S. Forest Service. Most of the vegetatal material within the reservoir consists of trees, with very little in brush. The trees are quite small, and the stands are not very dense. In the Watana reservoir area, an estimated 18,000,000 cubic feet of wood exists averaging approximately 500 cubic feet of low commercial quality, and some very significant logging problems would be posed by the steep slopes and incised terrain encountered in the area. Approximately 87 percent of the available timber are soft woods. The results of the timber reconnaissance studies are described in more detail in Appendix C3.

The combination of steep terrain, moderate-light tree stocking levels, small trees, erosive potential of the reservoir slopes, remoteness, and very restricted access to the reservoirs are major factors affecting the choice of harvesting systems to be utilized for this project. Such systems include high lead, skyline, tractor, whole tree logging with or without chippers, balloon and helicopter. Each system has its own advantages and disadvantages and set of conditions under which its used is optimized.

Present market demand for the timber at Susitna is low, however the worldwide demand wood fluctuates considerably. It is anticipated that use of the harvested material would be limited to either sale as wood - waste products and as fuel.

Slash material including brush and small trees, which will be unsuitable for either of the above uses, will be either burned in a carefully controlled manner consistent with applicable laws and regulations, or hauled to a disposal site in and adjacent to the reservoir. Material placed in disposal areas will be covered with a earthfill cover sufficient to prevent erosion and subsequent exposure.

12.15 - Tailrace

Two tailrace pressure tunnels will be provided at Watana to carry water from the surge chamber to the river. The tunnels will have a modified diameter of horseshoe cross-section with a major internal dimension of 35 feet. For preliminary design they are assumed to be fully concrete-lined throughout, with a minimum concrete thickness of 12 inches and a length of 1,800 feet.

The tailrace tunnels will be arranged to discharge into the river between the main dam and the main spillway. In view of the severe limitations on space in this area, one tailrace tunnel will be designed to discharge through one of the diversion tunnel portals. The cross section of the tailrace tunnel will be modified over the common length of 300 feet to the shape of the diversion tunnel in order not to impair the hydraulic performance of the tailrace tunnel. After commissioning, the diversion tunnel upstream section will be plugged with concrete.

The size of the two tailrace tunnels was selected after an economic study of the cost of construction and the capitalized value of average annual energy losses caused by friction, bends, and changes of section. In an emergency, however, the station can be operated using one tailrace tunnel, with increased head losses. For such an emergency condition, tailrace intake stoplog guides will be provided in the surge chamber. The surge chamber will be designed for full load rejection with either one or two tailrace tunnels in operation.

The tailrace portals will be reinforced concrete structures designed to reduce the outlet flow velocity, and hence the velocity head loss at the exit to the river. The minimum rock cover required above the tunnels will be 1.5 times the major excavated dimension (about 54 feet), and the portals will also provide the necessary transition length to the river where the rock cover would be less than 54 feet.

12.16 - Turbines and Generators

(a) Unit Capacity

The Watana powerhouse will have six generating units with a nominal capacity of 170 MW. This is the available capacity with minimum December reservoir level (El. 2112) and a corresponding gross head of 562 feet on the station.

The head on the plant will vary from 735 feet maximum (724 feet net head) to 595 feet minimum (584 feet net level). Because maximum turbine output varies approximately with the 3/2 power of head, the maximum unit output will change with head, as shown on Figure 12.21.

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).

The generator rating has been selected as 190 MVA with a 90 percent power factor, which corresponds to a power output of 170 MW. The generators will be capable of a continuous 15 percent overload; this will allow a unit output of 196 MW. At maximum reservoir water level, the turbines will be operated below maximum (full gate) output to avoid overloading of the generators.

(b) Turbines

The turbines will be of the vertical shaft Francis type with steel spiral casing and a concrete elbow-type draft tube. The draft tube will comprise a single water passage without a no center pier.

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 724 feet and 584 feet respectively. The full gate output of the turbines will be about 275,000 hp at 724 feet net head and 200,000 hp at 584 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 12.22.

A speed of 225 rpm has been selected for the unit for preliminary design purposes. The resulting turbine specific speed (N_s) is 32.4. As shown on Figure 12.23, this is within present day practice for turbines operating under a head of 670 feet. In general, a lower speed machine will increase the cost of the turbines and generators as well as the powerhouse civil cost because of the increased physical size of the generating equipment. A higher speed unit, on the other hand, requires a deeper unit setting and is generally considered to be a less conservative design with increased risk of vibration and rough operation. The difference in efficiency between the higher and lower speed machines at this head range is very small, with the increase in efficiency which is associated with a physically larger (lower speed) runner offset by higher disc friction and seal leakage losses. For an underground powerhouse, the incremental cost of increasing the unit setting is usually relatively inexpensive; therefore, assuming no change in efficiency, the trend in unit selection is to choose as high a speed as possible consistent with satisfactory precedent and good operating experience with similar specific speed turbines. Draft tube vortex and surge phenomena may also have an influence on the selection of unit speed as discussed below. The turbine data is summarized in Table 12.7.

On the basis of information from turbine manufacturers and the studies on the power plant layout, the centerline of the turbine distributor has been set at 30 feet below minimum tailwater level. The final setting of the unit will be established in conjunction with the turbine manufacturer when the contract for the supply of the turbine equipment has been awarded.

The turbines will be of conventional design, generally of welded or cast steel construction with forged steel shafts and pins. Because of the remote location of the project and the desired high reliability/availability of the equipment, special consideration should be given to reducing cavitation pitting on the turbines. This will include:

- Provision of weldable stainless steel runners;
- Careful profiling and finishing of the water passage surfaces of the runner wicket gates and stay ring;
- A conservative unit setting; and
- Extensive cavitation tests on the turbine model.

Bulkhead domes will be provided with two of the turbines (Units 3 and 4) to be installed at the bottom of the draft tube liner at the time of turbine installation. The domes permit work to continue on turbine installation after the tailrace, surge chamber, and draft tubes are flooded (prior to startup of Unit 1), without installing draft tube gates.

Because of the relatively short length of the intake penstock and a surge tank location immediately downstream of the powerhouse, the hydraulic transient characteristics of the turbines are favorable. Assuming normal generator inertia (H = 3.5 MW-Sec/MVA), a preliminary analysis has indicated the following:

- Water starting time (Tw) .... 1.6 seconds
- Mechanical starting time (Tm) .... 6.6 seconds
- Regulating ratio (Tm/Tw) 4.1
- Governor time .... 6.0 seconds
- Speed rise on full load rejection .... 42 percent
- Penstock pressure use on full load rejection .... 30 percent

The regulating ratio is above the minimum recommended by the USBR for good regulating capacity. Also, unit speed rise and penstock pressure rise are all well within normal accepted values. Because of the deep unit setting and the relatively short distance between the turbine and the tailrace surge tank, there will be no problems with draft tube water column separation.

The Watana project will form a large portion of the overall system generating capacity in Alaska; therefore, satisfactory operation of the units over a very wide range of loads will be important. Although there are Francis turbine installations which operate for considerable periods at very small gate openings, operation below about 50 percent load generally becomes increasingly rough because of reduced efficiency of the turbines.

The ability to operate at part load will also depend on the draft tube surge phenomena and associated generator power swings. The surging occurs on many turbines, particularly the Francis type, and normally has a frequency of about 1/3 to 1/5 of the turbine rotational speed. These surges, which may occur from about 30 percent to sometimes as high as 80 percent wicket gate opening, result in pulsating torque on the turbine runner and corresponding generator power fluctuations. The condition becomes more severe when the surge frequency closely corresponds to the natural frequency of the generator. To reduce the possibility of unacceptable power swings and unit vibration, it is desirable to have the surge frequency different from the penstock pressure wave frequency and the generator natural frequency.

The estimated natural frequency of the generators will be about 1.3 cycles/s, which is undesirable when considering a possible surge frequency of about 0.75 to 1.25 cycles/s.

The selection of a lower unit speed would separate the draft tube surge frequency from the generator natural frequency; however, this will increase the generating equipment as well as powerhouse cost. It may be possible to increase the natural frequency of the generator by reducing the generator inertia (WR^2) as much as possible; however, the lower WR^2 has an adverse effect on the transient characteristics of the unit and may affect electrical system stability.

Other measures which may be employed are:

- Careful model studies of the turbine to accurately predict draft tube vortex/surge phenomena. Altering the shape of the draft tube can reduce surge problems; however, a reduction of part-load efficiency may result.

- Provision for air admission to the draft tube. This is done on more or less a trial and error basis and may include injection of air from the station 100 psi compressed air system, special low pressure compressors provided specifically for draft tube air admission, and/or provision of an "air-head" which allows atmospheric air to be drawn down the generator and turbine shaft and through the runner cone.

- Additions of fins to the draft tube cone immediately below the runner. While this has been used on many installations, there have been instances of structural failure of the fins.

Another approach currently under study by the USBR is to damp the power oscillations resulting from the draft tube surges by varying the generator excitation. Computer simulation indicates that is feasible to dampen the large oscillations; however, this has yet to be tested in a prototype unit.

Employing one or more of the above approaches, a design may be achieved that operates satisfactorily without serious generator power swings. The potential problem, however, must be given serious consideration in the design stages.

(c) Generators

(i) Type and Rating

The six generators in the Watana powerhouse will be of the vertical shaft, overhung type directly connected to the vertical Francis turbines. The arrangement of the units is shown in Plates ___ and the single line diagram is shown in Plate ___.

The optimum arrangement at Watana will consist of two generators per transformer bank, with each transformer bank comprising three single-phase transformers. (Development of this scheme is described in Section 12.18). The generators will be connected to the transformers by isolated phase bus through generator circuit breakers directly connected to the isolated phase bus ducts.

Each generator will be provided with a high initial response static excitation system. The units will be controlled from the Watana surface control room, with local control facility also provided at the powerhouse floor. The units will be designed for black start operation.

The generators are rated as follows:

Rated Capacity:	190 MVA, 0.9 power factor
Rated Power:	170 MW
Rated Voltage:	15 kV, 3 phase, 60 Hertz
Synchronous Speed:	225 rpm
Inertia Constant:	3.5 kW-sec/kVA
Transient Reactance:	28 percent (maximum)
Short Circuit Ratio:	1.1 (minimum)
Efficiency at Full Load:	98 percent (minimum)

The generators will be of the air-cooled type, with water-cooled heat exchangers located on the stator periphery. The ratings given above are for a temperature rise of the stator and rotor windings not exceeding 60°C with cooling air at 40°C.

The generators will be capable of delivery 115 percent of rated MVA continuously (195.5 MW) at a voltage of ±5 percent without exceeding 80°C temperature rise in accordance with ANSI Standard C50.10.

The generators will be capable of continuous operation as synchronous condensers when the turbine is unwatered, with an underexcited reactive power rating of 140 MVAR and an overexcited rating of 110 MVAR. Each generator will be capable of energizing the transmission system without risk of self-excitation.

The design data of the generators stated above should be reviewed during the detailed design stage for overall economic and technical design and performance requirements of the power plant and the power system.

(ii) Generator ~~Construction~~

The generator will be of a modified umbrella type overhung construction, with a combined thrust and guide bearing below the rotor and a guide bearing above the rotor. The lower bearing bracket will support the rotor and turbine runner weights and the unbalanced hydraulic thrust of the runner. All removable parts, including turbine parts, will be designed for removal through the generator stator.

Approximate dimensions and weights of the principal parts of the generator are given below:

Stator pit diameter	36 feet
Rotor diameter	22 feet
Rotor length (without shaft)	7 feet
Rotor weight	385 tons
Total weight	660 tons

It should be noted that these are approximate figures and they will vary between manufacturers, sometimes considerably. However, at this stage of design feasibility and planning, the dimensions and weights are considered appropriate and representative.

The generator stator windings will be insulated with Class B insulation as defined by ANSI Standard C50.10, of epoxy resin bonded type. The stator windings will be wye-connected for grounded operation through a neutral grounding transformer located in the generator neutral cubicle. The stator windings and laminated core will be shop-assembled in three or four sections for facility of transport and erection in the powerhouse.

The rotor will be designed to safely withstand the maximum runaway speed of the turbine. The rotor hub, yoke, and laminated rim will be designed for assembly at the powerhouse. The assembled rotor will be erected in the generator pit without the shaft, thereby requiring minimum crane lift and a considerable reduction of the powerhouse cavern height.

The rotating parts of the generator and turbine will be designed so that the critical speed exceeds the runaway speed of the unit by at least 20 percent. The design of the plant and power system will not require additional inertia in the rotating parts; the inertia constant specified thus will correspond to the "natural" inertia of the machine. Damper windings of low resistance and rugged construction will be provided on the pole shoes and designed to provide adequate damping currents for stabilized operation.

The thrust bearing will be of the adjustable shoe (Kingsbury) or precompressed spring (General Electric) type, oil-cooled, with high pressure oil injection during starting and stopping.

(iii) Generator Excitation System

The generator will be provided with a high initial response type static excitation system supplied with rectified excitation power from transformers connected directly to the generator terminals. The excitation system will be capable of supplying 200 percent of rated excitation field (ceiling voltage) with a generator terminal voltage of 70 percent. The power rectifiers will have a one-third spare capacity to maintain generation even during failure of a complete rectifier module.

The excitation system will be equipped with a fully static voltage regulating system maintaining output from 30 percent to 115 percent, within ± 0.5 percent accuracy of the voltage setting. Manual control will be possible at the excitation board located on the powerhouse floor, although the unit will normally be under remote control, as described in Section 12.18 covering the control systems of the plant.

The static excitation system will also include maximum and minimum limiters and reactive current compensator, and will be suitable for parallel joint control of the units. Field flashing during startup will be from the 125 volt dc station battery.

(iv) Erection and Tests

As is normal for large hydroelectric generators, the machines will not be assembled completely and tested in the factory. The erection and tests of the generators at the powerhouse, therefore, will assume greater importance in the successful commissioning of the station and should be carefully coordinated with that of the turbines and civil works.

The assembly of the stator sections will be done in the pit. The rotor will be assembled in the erection bay. The powerhouse crane will be capable of lifting the completed rotor assembly and lowering it into the stator, and onto the thrust bearing and shaft assembly on the bracket supports. Alignment and tests of the rotor, turbine runner, and shaft will be done to tolerances specified in NEMA/ANSI Standards.

The generators will be fully tested after assembly and mechanical run tests, including dielectric tests, saturation tests, heat run, efficiency, and full-load rejection tests. Ceiling voltage and response of the excitation system will be tested. Operation of the unit within specified vibration limits will be checked.

(d) Governor System

The governor system which control the generating unit will include a governor actuator and a governor pumping unit. A single system will be provided for each unit. The governor system operating pressure will be 600 to 1000 psi, as recommended by the governor system manufacturer.

The governor actuator will be the electric hydraulic type and will be connected to the computerized station control system. The governor pumping unit will include governor pumps, an accumulator tank, and a sump tank. Each unit will have three governor pumps: two main pumps which operate intermittently, and one jockey pump which operates continuously while the turbine wicket gates are open and intermittently when the gates are fully closed.

12.17 - Miscellaneous Mechanical Equipment

(a) Powerhouse Cranes

Two overhead traveling bridge type powerhouse cranes will be installed in the powerhouse. The cranes will be used for:

- Installation of turbines, generators, and other powerhouse equipment; and

- Subsequent dismantling and reassembly of equipment during maintenance overhauls.

The cranes may also be used by the civil construction contractor for powerhouse construction. Alternatively, the civil contractor will provide a separate crane which will use the same runway as for the main powerhouse cranes.

Each crane will have a main and auxiliary hoist. The combined capacity of the main hoist for both cranes will be sufficient for the heaviest equipment lift, which will be the generator rotor, plus an equalizing beam. A tentative crane capacity of 205 tons has been established. The auxiliary hoist capacity will be about 25 tons.

The powerhouse cranes will be cab controlled. Consideration may also be given to providing radio control for the cranes.

(b) Draft Tube Gates

Draft tube gates will be provided to permit dewatering of the turbine water passages for inspection and maintenance of the turbines. The draft tube gate openings (one opening per unit) will be located in the surge chamber. The gates will be of the bulkhead type, installed under balanced head conditions using the surge chamber crane described below. Four gates have been assumed for the six units, with each gate a single leaf, 20 feet by 20 feet.

When Unit 1 is ready for startup, the gates will be installed in Units 2, 5, and 6, with one gate available for Unit 1. Turbine bulkhead domes will be installed in Units 3 and 4.

(c) Surge Chamber Gate Crane

A crane will be installed in the surge chamber for installation and removal of the draft tube gates as well as the tailrace tunnel intake stoplogs. The crane will either be a monorail (or twin monorail) crane, a top running crane, or a gantry crane. For the preliminary design, a twin monorail crane has been assumed. The crane will be about 45 tons in capacity, pendant operated, and will have a two point lift. A follower will be used with the crane for handling the gates and stoplogs. The crane will normally travel along the upstream side of the surge chamber; however, the crane runway will have a transfer mechanism for moving the crane to the downstream side of the surge chamber for installation or removal of the tailrace tunnel intake stoplogs. The crane runway will extend over the tailrace tunnel stoplog storage area at one end of the surge chamber.

(d) Miscellaneous Cranes and Hoists

In addition to the powerhouse cranes and surge chamber gate crane, the following cranes and hoists will be provided in the power plant:

- A 5-ton monorail hoist in the transformer gallery for transformer maintenance;

- A 4-ton monorail hoist in the circuit breaker gallery for handling the main circuit breakers;

- Small overhead jib or A-frame type hoists in the machine shop for handling material; and

- A-frame or monorail hoists for handling miscellaneous small equipment in the powerhouse.

(e) Elevators

Access and service elevators will be provided for the power plant as follows:

- An access elevator from control buildings to powerhouse;
- A service elevator in the powerhouse service bay; and
- Inspection hoists in the cable shafts.

For preliminary design purposes, a 12,000-lb, double-deck elevator has been assumed for access to the powerhouse from the control building. The elevator will be located in the access shaft and will travel at a speed of about 500 ft/min; it will be operated by a friction type hoist located above the elevator shaft. The elevator will have a single landing at the control building plus four underground landings.

The service elevator in the powerhouse service bay will have a capacity of 2,000 to 4,000 pounds and will provide access to the various powerhouse floors. The elevator will travel at about 100 to 150 ft/min and will be operated either by a friction hoist or a hydraulic cylinder.

"Alimak" type rack and pinion man hoists have been assumed for the cable shafts, to use in inspection and/or maintenance of the oil-filled cables and control cables. The hoist would also provide emergency access from the power plant. Each hoist will have a capacity of about 900 pounds and travel at a speed of approximately 130 ft/min.

(f) Power Plant Mechanical Service Systems

The mechanical service systems for the power plant can be grouped into six major categories:

- Station water systems;
- Fire protection;
- Compressed air;
- Oil storage and handling;
- Drainage and dewatering; and
- Heating, ventilation and cooling.

(i) Station Water Systems

The station water systems will include the water intake, cooling water systems, turbine seal water systems, and domestic water systems.

The water intakes will supply water for the various station water systems in addition to fire protection water. The water can be taken from the penstock; however, pressure-reducing valves will be necessary because of the high pressure of the water (about 330 psig maximum). Alternatively, water can be supplied from the draft tube using pumps to provide suitable pressure. For preliminary design purposes, the latter approach has been adopted with a water intake at each draft tube. The water will pass through an automatic backwash strainer which will limit the maximum particle size in the water to about 1/16 inch. An interconnecting header will permit a strainer to be taken out of service without affecting operation of a generating unit. Each strainer will be sized to handle the water requirements for two units.

On a unit basis, cooling water will be required for generator air coolers, turbine and generator bearing coolers, transformers, and powerhouse unit air coolers. The total cooling water requirements for each unit will be about 4,000 gpm. In addition, the compressed air systems in the service bay will require approximately 100 gpm of cooling water. One cooling water pump will be provided per unit which will take water from downstream from the water intake strainer. To ensure suitable reliability, the cooling water pumps for two units will be interconnected, with each pump capable of handling the flow for both units. Two cooling water pumps in the service bay will handle compressor cooling water requirements. The cooling water for each unit will discharge into the turbine draft tube, while the compressor cooling water will flow into the station drainage system.

Turbine seal water will be supplied to the seal on the main shaft and to the runner seals when the unit is spinning in air (i.e., in spinning reserve mode). Filtered water may or may not be required, depending on the type of shaft seal. If no filtration is needed, the seal water will be taken directly from the high-pressure side of the cooling water pumps. If filtration is necessary, a single system will be provided for the powerhouse. The system will have two filters and two pumps which will take water from downstream from the water intake strainer and distribute the water to each unit via a looped header.

Domestic water will be required for the washrooms, lunch rooms, drinking fountains, and a service sink and emergency eyewash in the battery room. Peak domestic water requirements are expected to be about 30 gpm. The system will have two pumps and a hydropneumatic tank. Water will be taken from the water intake system and will be treated by chlorination or other means as necessary.

(ii) Fire Protection System

The power plant fire protection system will consist of a fire protection water system with fire hose stations located throughout the powerhouse and transformer gallery; sprinkler systems for the generators, transformers, and the oil rooms; and portable fire extinguishers located in strategic areas of the powerhouse and transformer gallery. Carbon dioxide could be used in the generator rather than a sprinkler system; however, the water system is recommended because of the safety hazard of CO_2.

Fire protection water will be taken from the station water intakes. Pressurized water will be provided by a pumped system with two main fire pumps as well as a jockey pump, or alternatively by a head tank with two supply pumps which keep the head tank full. For preliminary design purposes, a system with a head tank has been selected because of the increased reliability of the system. With an underground powerhouse, a head tank can be provided quite easily at a suitable elevation as an adit to the access shaft.

The capacity of the head tank will be about 100,000 gallons; the tanks will have two compartments to permit draining of half the water for inspection and maintenance. For reliability, the water supply pumps will have two electrical power sources.

Fire hose stations will be provided on all floors of the powerhouse, in the transformer gallery, and in the bus tunnels. Service water outlets will be installed at the various fire hose stations to supply water for washing downs floors or equipment. The sprinkler systems for generators, transformers, and oil rooms will be the dry deluge type, operated by a solenoid valve which in turn will be activated by detectors in the respective area.

The portable fire extinquishers will generally be carbon dioxide or a dry chemical type.

(iii) Compressed Air Systems

Compressed air will be required in the powerhouse for the following:

- Service air;
- Instrument air;
- Generator brakes;
- Draft tube water level depression;
- Air blast circuit breakers; and
- Governor accumulator tanks.

For the preliminary design, two compressed air systems have been assumed: a 100-psig air system for service air, brake air, and air for draft tube water level depression; and a 1,000-psig high-pressure air system for governor air and circuit breaker air. For detailed plant design, a separate governor air system and circuit-breaker air system may be provided.

The service air systems will have three air compressors of the rotary screw or reciprocating type, each with a capacity of about 200 cfm. The system will have four air receivers, two with approximately 800 ft^3 capacity used for the draft tube water level depression system, and two with approximately 150 ft^3 used for service and brake air. The system will be designed to give priority to the brake air system. Service air piping with air hose stations will be located on all floors of the powerhouse and in the transformer gallery.

The high-pressure governor/circuit-breaker air system will have three reciprocating air compressors with approximately 30-cfm capacity each, and three small air receivers. The governor air system will supply air for initial filling of the governor system accumulator tanks and for makeup air to replace air lost through leakage and air dissolved in the governor system oil.

The circuit breaker air system will provide compressed air for operation of the main breakers. To insure dry air for the breakers, the air will be stored at 1,000 psig and then reduced to about 350 psig for operation of the breakers.

Instrument air will also be taken from the high-pressure air system.

(iv) Oil Storage and Handling

Facilities will be provided for replacing oil in the transformers and for topping-up or replacing oil in the turbine and generator bearings and the governor pumping system. For preliminary design purposes, two oil rooms have been assumed, one in the transformer gallery and one in the powerhouse service bay.

The transformer gallery will have two oil storage tanks, one for filtered oil and the other for unfiltered oil. Each tank will have a capacity at least equal to the volume of oil in one transformer (about 8,000 gallons). A header with valve stations at each transformer will be used for transferring oil to and from the transformers. Oil will be transferred by a portable pump and filter unit.

A similar system will be provided in the powerhouse with a filtered and unfiltered oil tank and distribution header with valve stations at each unit. The oil tank capacity will be equal to the total oil volume for one unit (about 3,000 gallons).

During the detailed design stages, consideration should be given to the use of mobile oil tanks located in a parking area near the powerhouse and transformer gallery, near the access tunnel.

(v) Drainage and Dewatering Systems

The drainage and dewatering systems will consist of:

- A unit dewatering and filling system;
- A clear water discharge system; and
- A sanitary drainage system.

The dewatering and filling systems will consist of two sumps each with two dewatering pumps and associated piping and valves from each of the units. To prevent station flooding, the sump will be designed to withstand maximum tailwater pressure. For preliminary design purposes, submersible dewatering pumps have been assumed. Vertical turbine type pumps can also be considered; however, since the dewatering system acts as an emergency drainage system, the pump columns would have to be extended so that the motors are above maximum tailwater level. Another option is turbine-driven pumps, but these are generally very costly. A valved draft tube drain line will connect to a dewatering header running along the dewatering gallery. The spiral case will be drained by a valved line connecting the spiral case to the draft tube. Suitable provisions will be necessary to insure that the spiral case drain valve is not open when the spiral case is pressurized to headwater level. The dewatering pump discharge line will discharge water into the surge chamber. The general procedure for dewatering the unit will be to close the intake gate, drain the penstock to tailwater level through the unit, then open the draft tube and spiral case drains to dewater the unit. Unless the drainage gallery is below the bottom of the draft tube elbow, it will not be possible to completely unwater the draft tube through the unwatering header. If necessary, the remainder of the draft tube can be unwatered using a submersible pump lowered through the draft tube access door. Unit filling to tailwater level will be accomplished from the surge chamber through the dewatering pump discharge line (with a bypass around the pumps) and then through the draft tube and spiral case drain lines. Alternatively, the unit can be filled to tailwater level through the draft tube drain line from an adjacent unit. Filling the unit to headwater pressure will be accomplished by "cracking" the intake gate and raising it about 2 to 4 inches.

The clearwater drainage system will handle normal drainage into the power plant. Drainage will be collected by a network of floor drains, trench drains, pressure relief drains, and equipment drains which discharge into gravity drainage sumps where items are pumped to the surge chamber. The station will have three main sumps, two in the powerhouse adjacent to the dewatering sumps and one in the transformer gallery. Smaller sumps will be located in appropriate areas such as the elevator pits and the upstream drainage gallery.

The sumps in the powerhouse will have submersible pumps for the same reasons as discussed above for the dewatering system. The transformer gallery will have vertical turbine type pumps. The drainage sumps in the powerhouse will have an overflow line which will discharge water into the adjacent dewatering sump should inflow into the drainage sumps exceed the capacity of the drainage pumps. The overflow line will have a flap valve to prevent reverse flow from the dewatering sump.

Particular care will be taken to prevent accidental oil spills from being discharged into the powerhouse. The following provisions will be made:

- All three main sumps will have oil contamination detectors to obtain the pressure of oil in the sumps;

- Drainage into the sumps will first pass through an oil separator;

- Controls for the drainage pumps into the transformer gallery will be interlocked with the transformer fire protection sprinkler system. Activation of the sprinklers, which signifies a transformer fire and the possibility of a major oil spill, will prevent the drainage pumps from starting until the drainage sump is almost full. It will be possible to retain about 40,000 gallons of oil/water in the sump before the pump start (each transformer holds about 8,000 gallons of oil). In this manner, it will be possible to retain a large amount of oil in the sump where it may be skimmed off; and

- Suitable oil retention curbs will be provided in the oil rooms.

Sanitary drainage from the washrooms, lunch room, and drinking fountains will drain to a packaged sewage treatment plant and then will be discharged into the surge chamber via sewage lift pumps.

(vi) Heating, Ventilation and Cooling

The heating and ventilation system for the underground power plant will be designed primarily to maintain suitable temperatures for equipment operation and to provide a safe and comfortable atmosphere for operating and maintenance personnel. Air will be drawn into the power facilities through one or more shafts or tunnels, circulated throughout the power plant, and discharged from the power plant through other shafts and tunnels. For preliminary design purposes it has been assumed that air will be drawn down the access and the cable shafts, and discharged out through the access tunnel; however, the actual arrangement will depend upon the final design.

The power plant will be located in mass rock which has a constant year around temperature of about 40°F. Considering heat given off from the generators and other equipment, the primary requirement will be for air cooling. Initially, some heating will be required to offset the heat loss to the rock, but after the first few years of operation an equilibrium will be reached with a powerhouse rock surface temperature of about 60 to 70°F.

Air cooling will be accomplished by providing suitable air changes incorporating cooling coils in the air circulation system. Cooling water from the station service water supply will be circulated through the cooling coils. In winter, some heating may be required to moderate the temperature of the incoming air into the power plant. Allowance must be made in the design for the possibility that large quantities of air (up to about 6,000 cfm per unit) may be required for turbine aeration.

Other factors which must be considered or incorporated in the design are:

- To prevent or minimize the circulation of combustion products in the event of a fire, powerhouse ventilation should be separate from transformer gallery ventilation and provision should be made for isolating the two areas; and

- Suitable air locks will be necessary to preclude adverse chimney effects in the shafts.

(g) Surface Facilities Mechanical Service Systems

The mechanical services at the control building on the surface will include:

- A heating, ventilation, and air conditioning system for the control room;
- Domestic water and washroom facilities; and
- A halon type fire protection system for the control room.

Domestic water will be supplied from the powerhouse domestic water system, with pumps located in the powerhouse and piping up through the access shaft. Sanitary drainage from the control building will drain to the sewage treatment plant in the powerhouse through piping in the access tunnel.

The standby generator building will have the following services:

- A heating and ventilation system;

- A fuel oil system with buried fuel oil storage tanks outside the building, and transfer pumps and a day tank within the building; and

- A fire protection system of the carbon dioxide or halon type.

(h) Machine Shop Facilities

A machine shop and tool room will be located in the powerhouse service bay area with sufficient equipment to take care of all normal maintenance work at the plant, as well as machine shop work for the larger components at Devil Canyon. For preliminary design purposes, an area of about 1,500 ft^2 has been allocated for the machine shop and tool room. The actual equipment to be installed in the machine shop will be decided during the design stages of the project; however, it will generally include drill presses, lathes, a hydraulic press, power hacksaw, shaper, and grinders.

12.18 - Accessory Electrical Equipment

The accessory electrical equipment described in this section includes the following:

. Main generator step-up 15/345 kV transformers;
. Isolated phase bus connecting the generator and transformers;
. Generator circuit breakers;
. 345 kV oil-filled cables from the transformer terminals to the switchyard;
. Control systems of the entire hydro plant complex; and
. Station service auxiliary AC and DC systems.

Other equipment and systems described include grounding, lighting system, and communications.

The main equipment and connections in the power plant are shown in the single line diagram, Plate 60A. The arrangement of equipment in the powerhouse, transformer gallery, and cable shafts is shown on Plates 57 through 59.

(a) Selection of Transformers and H.V. Connections

(i) General

Nine single-phase transformers and one spare transformer will be located in the transformer gallery. Each bank of three single-phase transformers will be connected to two generators through generator circuit breakers by isolated phase bus located in individual bus tunnels. The HV terminals of the transformer will be connected to the 345 kV switchyard by 345 kV single-phase oil-filled cable installed in 700-foot-long vertical shafts. There will be two seats of three single-phase 345 kV oil-filled cables installed in each cable shaft. One set will be maintained as a spare three phase cable circuit in the second cable shaft. These cable shafts will also contain the control and power cables between the powerhouse and the surface control room, as well as emergency power cables from the diesel generators at the surface to the underground facilities.

A number of considerations led to the choice of the above optimum system of transformation and connections. Different alternative methods and equipment designs were also considered. In summary, these are:

- One transformer per generator vs one transformer for two generators;

- Underground transformers vs surface transformers;

- Direct transformation from generator voltage to 345 kV vs intermediate step transformation to 230 kV or 161 kV, and then to 345 kV;

- Single-phase vs three-phase transformers for each alternative method considered; and

- Oil-filled cable vs solid dielectric cable for SF6 gas-insulated bus.

(ii) Reliability Considerations

Reliability considerations will be based on the general reliability requirements for generation and transmission described in Section 15 regarding the forced outage of a single generator, transformer, bus or cable in addition to planned or scheduled outages in a single contingency situation, or a subsequent outage of equipment in the double contingency situation. The system should be capable of re-adjustment after the outage for loading within normal ratings and for loading within emergency ratings.

The generators will be rated with a 115 percent continuous overload capability. All main connections and equipment including the transformers, circuit breakers, isolated phase bus, and 345 kV cables will be rated for continuous operation at the 115 percent overload rating of the generators.

Emergency ratings are different for different items of equipment and emergency periods. It generally varies between 110 to 130 percent in summer to 120 to 140 percent in winter for a 4 to 12 hour period, with somewhat higher values for very short (1 hour) emergency periods.

(iii) Technical and Economic Considerations

The use of surface transformers connected directly to the underground generators by isolated phase bus was ruled out at the outset due to significantly higher costs and higher losses associated with generator isolated phase buses. The incremental cost could be decreased if three units were connected to one transformer, but such a compromise is not acceptable due to reliability considerations.

In general, 3-phase transformers are preferred to single-phase transformers because of their lower overall costs, smaller overall dimensions and smaller underground gallery dimensions. However, transport limitations seriously affect the use of the larger size 3-phase transformers, both in dimensions and weight. The following are the road and rail data available:

- Parks and Denali Highways

  Maximum load - 150,000 lb
  Overweights require special permits.

- Railway

  Maximum Weight - 263,000 lb
  Dimension Limits - 16 feet high, 10 feet wide

A further check of these design limitations for the selected sizes of transformers is recommended during the detailed design stage. A careful route reconnaissance study is also required.

Single-phase transformers are therefore recommended for the 6-unit power plant. The grouped unit arrangement with two generators per transformer will allow a smaller gallery length, with center-to-center spacing comparable to the generator spacing. The grouped unit arrangement is the recommended arrangement. The alternative with one transformer per generator will requie a gallery about 300 feet longer.

One distinct advantage of single-phase transformers is that a spare transformer can be provided at a fairly low incremental cost.

The double-step transformation scheme (15/161 KV generator-transformer, 161 KV cable and 161/345 KV auto-transformer at the switchyard) is economicallly competitive with the direct transformation scheme (15/345 KV), resulting from a number of tradeoffs: cost/MVA per transformer is lower; also dimensions, weights and cavern dimensions are lower; but the intermediate-voltage transformer costs are additional.

Direct transformation (15/345 KV) is better from system transient stability viewpoint since the overall impedance of the generator unit to the 345 KV bus is lower. Furthermore, it has a better overall reliability since there is no one less voltage level and, therefore, less equipment in the generating "chain" of equipment. This scheme costs about $2 million less in overall costs compared to the double-step transformation scheme.

The comparison between 345 KV oil-filled cables and other 345 KV cable and bus system is made in Section 12.18. The SF6 bus is about 5 to 6 times the cost of the oil-filled cables. It also requires a larger diameter cable shaft. The oil-filled cable is well proven at a number of underground power installations and was therefore selected for both technical and economic considerations.

(c) Main Transformers

(i) Rating and Characteristics

The nine single-phase transformers (three transformers per group of two generators) and one spare transformer, will be of the two winding, oil-immersed, forced-oil water-cooled (FOW) type, with rating and electric characteristics as follows:

Rated capacity:	145 MVA
High voltage winding:	345 / 3 kV, Grounded Y
Basin insulation level (BIL) of H.V. winding:	1300 kV
Low voltage winding:	15 kV, Delta
Transformer impedance:	15 percent

The temperature rise above air ambient temperature of 40°C is 55°C for the windings for continuous operation at the rated kVA.

(ii) Construction

The transformers will be of the FOW type with water-cooled heat exchangers which remove the heat from the oil circulating through the windings. A one-third spare cooler capacity will be provided. The transformer will be of the forced oil directed type with a design aimed to achieve minimum dimensions and weight for shipping purposes. The low voltage terminals will be connected to the isolated phase bus, and the high voltage terminals to the 345 kV oil-filled cable box termination at the transformer.

Lightning arresters will be connected directly to the high voltage terminals. The transformer installation in the gallery will be designed to provide the necessary ground and safety clearances from the live 345 kV terminals to all nearby equipment and structures.

The tank underbase will be provided with flanged wheels for transport on rails. The spare single-phase transformer will be exactly identical to the remaining nine single-phase transformers. It will be maintained in a state of maximum readiness, for connection in the shortest practical time to replace any of the main transformers.

The transformers will be fully tested and inspected in the factory according to ANSI/NEMA Standards. They will be shipped without oil and filled with inert gas for protection. At the site, erection would be mainly for external fittings such as bushings, lightning arresters, heat exchangers, piping, and electrical connections.

(iii) Fire Protection

Fire walls will separate each single-phase transformer. Each transformer will be provided with fog-spray water fire protection equipment, automatically operated from heat detectors located on the transformer.

(d) Generator Isolated Phase Bus

(i) Ratings and Characteristics

The isolated phase bus main connections will be located between the generator, generator circuit breaker, and the transformer.

Tap-off connections will be made to the surge protection and potential transformer cubicle, excitation transformers, and station service transformers. Bus duct ratings are as follows:

	Generator Connection	Transformer Connection
Rated current, amps	9,000	18,000
Short circuit current momentary, amps	240,000	240,000
Short circuit current, symmetrical, amps	150,000	150,000
Basic insulation level, kV (BIL)	150	150

The bus conductors will be designed for a temperature rise of 65°C above 40°C ambient temperature.

(ii) Construction

The bus will be of standard self-cooled design with conductor and tubular enclosure of aluminum. The current rating is such that either a self-cooled or forced cooled design will be possible. With a forced cooled design, the size and costs will be lower; however, if the forced-cooling plant fails, the bus would be severely derated to a rating less than 50 percent of the forced cooling rating. The self-cooled designs are used up to 30,000 amps rated current and are therefore recommended for this installation where the ratings will not exceed 18,000 amps.

The enclosure will be of welded construction and each bus will be grounded. The construction is highly reliable; will eliminate phase-to-phase faults, neutralize the magnetic field outside the enclosure, and provide protection against contamination and moisture, with consequent minimum maintenance requirements.

(e) Generator Circuit Breakers

The generator circuit breakers will be of the enclosed air circuit breaker design suitable for mounting in line with the generator isolated phase bus ducts. They are rated as follows:

Rated Current:	9,000 Amps
Voltage:	23 kV class, 3-phase, 60 Hertz
Breaking capacity, symmetrical, amps	150,000

The short circuit rating is tentative and will depend on detailed analysis in the design stage.

The breakers will be designed and constructed with a high degree of reliability. The phase spacing of the breakers will be generally the same as the isolated phase bus duct. The breakers will be mounted on strong foundations on the generator floor designed to absorb the reaction forces when the breaker operates. A separate compressed air plant will be provided for the high reliability compressed air system requirements of the air circuit breakers.

(f) 345 kV Oil-Filled Cable

(i) General

The recommended 345 kV connection is a 345 kV oil-filled cable system between the high voltage terminals of the transformer and the surface switchyard. The cable will be installed in a vertical cable shaft. Cables from two transformers will be installed in a single cable shaft.

This system of 345 kV connection was chosen after a technical and economic analysis of alternative methods of connection, including:

- SF6 isolated bus system;
- High pressure oil pipe cable system; and
- Solid dielectric cable system.

The SF6 bus system is considered to be the best alternative to the oil-filled cable system. Its advantages are a generally better overall reliability, including a low fire hazard. However, it costs approximately 5 to 6 times that of the oil-filled cable installation, and requires almost twice the diameter cable shaft of the cable installation. The overall cost difference is approximately $7,000,000 in direct costs.

The oil pipe cable will consist of three conductors contained within an oil-filled steel pipe. This system has the highest potential fire hazard of all the cable systems and is not recommended for high head vertical cable installations. The solid dielectric (polymeric) cables are still under development at the 345 kV to 500 kV voltage class.

It is recommended that further detailed study of the oil-filled cable in comparison with the SF6 bus and other more recent SF6 cable designs under development be undertaken at the design stage.

By far the greatest number of high voltage, high capacity installations utilize oil-filled cables. A formidable experience record is evident for the oil-filled cable installations associated with large power plants all over the world. Typical installations include the 525 kV/650 MVA units at Grand Coulee III, the 345 kV/550 MVA units at Churchill Falls in Canada, the 400 kV/2640 MVA cables at Severn River crossing in Great Britain, and the 400 kV/2340 MVA cables at Dinorwic pumped storage plant in Great Britain.

(ii) Rating and Characteristics

The cable will be rated for a continuous maximum current of 800 amps at 345 kV ± 5 percent. The maximum conductor temperature at the maximum rating will be 70°C over a maximum ambient of 35°C. This rating will correspond to 115 percent of the generator overload rating. The normal operating rating of the cable will be 87 percent, with a corresponding lower conductor temperature which will improve the overall performance and lower cable aging over its project operating life. Depending on the ambient air temperature, a further overload emergency rating of about 10 to 20 percent will be available during winter conditions.

The cables will be of single-core construction with oil flow through a central oil duct within the copper conductor. Cables will have an aluminum sheath and PVC oversheath. No cable jointing will be required for the 700 to 800 feet length cable installation.

(g) Control Systems

(i) General

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 _____. The control center will be linked through the supervisory system to the Central Dispatch Control Center at Willow as described in Section 14.

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 unit.

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 Watana and Devil Canyon plants will be capable of "black start" operation in the event of a complete black out or collapse of the power system. The control systems of the two plants and the Susitna Area Control Center complex will be supplied by a non-interruptible power supply.

(ii) Unit Control System

The unit control system will permit the operator to initiate an entire sequence of actions by pushing one button at the control console, provided all preliminary plant conditions have been first checked by the operator, and system security and unit commitment have been cleared through the central dispatch control supervisor. Unit control will be designed to:

- Start a unit and synchronize it with the system;
- Load the unit;
- Stop a unit;
- Operate a unit as running spare (runner in air with water blown down in turbine and draft tube); and
- Operate as a synchronous condenser (runner in air as above).

Unit control will be essentially possible at four different levels in a hierarchical organization of the control system:

- Local control at the machine floor at individual turbine-generator control boards (primarily designed for commissioning and recommissioning of units). It will be the responsibility of the operator for performing individual control operations in the correct sequence, and monitoring instrumentation during local control operations.

- Automatic or semi-automatic system for start-up and shut-down of generating unit at the local board at the machine floor.

- Fully automatic system at Susitna Area Control (at Watana) for Watana and Devil Canyon power plants. (This will be the normal Susitna operation.)

- Fully automatic system through supervisory control from Central Dispatch Center at Willow. (Abnormal or emergency situations only).

(iii) Computer-Aided Control System

Traditionally, control systems for power plants in general, and hydro plants in particular, have utilized hard-wired switchboard type equipment (such as electro-mechanical relays, instruments, alarm annunciators, signal lamps, mimic diagram and control switches) for the operation, indication, alarm and control of the power plant. Such equipment was installed both at the plant local control area on the machine floor as well as in the control room, with a limited degree of miniaturization of equipment at the control desks in the control room.

While traditional switchboard type equipment is still utilized at the local control level, supplemented with programmable control systems at many plants, the design of control and display equipment at modern central control rooms has been rapidly moving towards computer-aided or fully computer-controlled systems, especially where remote control operations are contemplated. One of the problems encountered by utilities is the necessity for operating personnel familiar with the conventional control systems to adapt to the new computer-aided control systems. In this contect, establishing a modern computer-aided control system in the Alaska Power Authority electrical system for the Susitna Project complex should not pose any special problems for the adaption and training of operators.

The computer-aided control system at the Susitna Area Control Center at Watana will provide for the following:

- Data acquisition and monitoring of unit (MW, MVAR, speed, gate position, temperatures, etc.);

- Data acquisition and monitoring of reservoir headwater and tailwater levels;

- Data acquisition and monitoring of electrical system voltage and frequency;

- Load-frequency control;

- Unit start/stop control;

- Unit loading;

- Plant operation alarm and trip conditions (audible and visual alarm on control board, full alarm details on VDU on demand);

- General visual plant operation status on VDU and on giant wall mimic diagram;

- Data logging, plant operation records;

- Plant abnormal operation or disturbance automatic recording; and

- Water management (reservoir control).

The block diagram of the computer-aided control system is shown in Plate ___. The supervisory control and telemetering system and central dispatch center system details are described in Section 14.

(iv) Local Control and Relay Boards

Local boards will be provided at the powerhouse floor equipped with local controls, alarms, and indications for all unit control functions. These boards will be located near each unit and will be utilized mainly during testing, commissioning, and maintenance of the turbines and generators. It will also be utilized as needed during emergencies if there is a total failure of the remote or computer-aided control systems.

The unit electrical protective relays will be mounted on relay boards, with one board for each generator located near the unit. Differential protection will be provided for each generator and transformer. The differential zones of protection overlap will include all electrical equipment and connections. The 345 kV oil-filled cable to the surface switchyard will be protected by a pilot-wire differential protection relay. The overall differential relay protects the generators, transformers, and 345 kV cable. Sensitive ground fault stator protection will be provided for the generator. Protection will also be provided for negative phase sequence operation, loss of excitation, overvoltage, and under frequency. A phase impedance relay will provide backup protection for the generator. Other protective relays are shown in Plate ___.

(v) Load-Frequency Control (Automatic Generation Control)

The load frequency control system will provide remote control of the output of the generator at Watana and Devil Canyon from the central dispatch control center through the supervisory and computer-aided control system at Watana. The basic method of automatic generation control (AGC) will use the plant error (differential) signals from the load dispatch center and will allocate these errors to the power plant generators automatically through speed-level motors. Provision will be made in the control system for the more advanced scheme of a closed-loop control system with digital control to control generator power.

The control system will be designed to take into account the digital nature of the controller-timed pulses as well as the inherent time delays caused by the speed-level motor run-up and turbine-generator time-constants.

The load set-point for the Susitna area generation will be set at the Central Dispatch Center. The summated power will be telemetered from the Susitna Area Control center to the Central Dispatch Center, from which the required differential plant generation ("error") will be determined and transmitted by the supervisory system to Susitna Area Control Center. From this point, the remaining functions for the automatic generation control will be carried out by the plant supervisory control systems to load the individual generating units at Watana and Devil Canyon.

The unit will be automatically removed from load-frequency control for various conditions including failure of supervisory system, unit controller or computer system, abnormally high plant frequency, unit shut-down, and dc power failure. When the unit is taken off automatic load-frequency control, it will be returned to manual load and frequency control by the operator at Watana Control room.

(h) Station Service Auxiliary AC and DC Systems

(i) Auxiliary AC System

The station service system will be designed to achieve a reliable and economic distribution system for the power plant and switchyard, in order to satisfy the following requirments:

- Station service power at 480 volts will be obtained from two 2,000 kVA auxiliary transformers connected directly to the generator circuit breaker outgoing leads of Units 1 and 3;

- Surface auxiliary power at 34.5 kV will be supplied by two separate 7.5/10 MVA transformers connected to the generator leads of Units 1 and 3;

- Station service power will be maintained even when all the units are shut down and the generator circuit breakers are open;

- 100 percent standby transformer capacity will be available;

- A spare auxiliary transformer will be maintained, connected to Unit 5; and

- "Black start" capability will be provided for the power plant in the event of total failure of the auxiliary supply system, 500 kW emergency diesel generators will be automatically started up to supply the power plant and switchyard with auxiliary power to the essential services to enable startup of the generators.

The main ac auxiliary switchboard will be provided with two bus sections separated by bus-tie circuit breakers. Under normal operating conditions, the station-service load is divided and connected to each of the two end incoming transformers. In the event of failure of one end supply, the tie breakers will close automatically. If both end supplies fail, the emergency diesel generator will be automatically connected to the station service bus.

Each unit will be provided with a unit auxiliary board supplied by separate feeders from the two bus sections of the main switchboard interlocked to prevent parallel operation. Separate ac switchboards will furnish the auxiliary power to essential and general services in the power plant.

The unit auxiliary board will supply the auxiliaries necessary for starting, running, and stopping the generator-turbine unit. These supplies will include those to the governor and oil pressure system, bearing oil pumps, cooling pumps and fans, generator circuit breaker, excitation system, and miscellaneous pumps and devices connected with unit operation.

The station essential service supplies will include powerhouse sump pumps, drainage pumps, compressors for circuit breakers, station air and generator brakes, dc battery chargers, control and metering devices, communications, fire protection pumps, and other miscellaneous essential power requirements.

The station general supplies will include powerhouse lighting, heating, ventilating and air-conditioning, elevators, cranes, machine shop and tools, and other miscellaneous pumps and general requirements.

The 34.5 kV supply to the surface facilities will be distributed from a 34.5 kV switchboard located in the surface control and administration building. Power supplies to the switchyard power intake, and spillway as well as the lighting systems for the access roads and tunnels will be obtained from the 34.5 kV switchboard.

The unit auxiliary board will supply the auxiliaries necessary for starting, running, and stopping the generator-turbine unit. These supplies will include those to the governor and oil pressure system, bearing oil pumps, cooling water pumps and fans, generator circuit breaker, excitation system, and miscellaneous pumps and devices connected with unit operation.

The station essential service supplies will include powerhouse sump pumps, drainage pumps, compressors for circuit breaker, air and generator brakes, dc battery chargers, control and metering devices, communications, fire protection pumps, and other miscellaneous essential power requirements.

The station general supplies will include powerhouse lighting, heating, ventilating and air-conditioning, elevators, cranes, machine shop and tools, and other miscellaneous pumps and general requirements.

The 34.5 kV supply to the surface facilities will be distributed from a 34.5 kV switchboard located in the surface control and administration building. Power supplies to the switchyard power intake, and spillway as well as the lighting systems for the access roads and tunnels will be obtained from the 34.5 kV switchboard.

The two 2000 kVA, 15000/480 volt stations service transformers and the spare transformer will be of the 3-phase, dry-type, sealed gas-filled design. The two 7.5/10 MVA, 15/34.5 kV transformers will be of the 3-phase oil-immersed OA/FA type.

Emergency diesel generators, each rated 500 kW, will separately supply the 480 volt and 34.5 kV auxiliary switchboards during emergencies. Both diesel generators will be located in the surface control building.

An uninteruptible high security power supply will be provided for the computer control system.

(ii) DC Auxiliary Station Service System

The dc auxiliary system will supply the protective relaying, supervisory, alarm, control, tripping and indication circuit in the power plant. The generator static excitation system will be started with "flashing" power from the dc battery. It will also supply the emergency lighting system at critical plant locations.

Separate duplicate lead-acid batteries for 125 volt dc will be provided in the powerhouse. The 48 volt battery supply for the supervisory and computer aided control system and microwave communications will be located in the surface control building.

The main battery system will be supplied by double charging equipment consisting of a full wave rectifier system with regulated output voltage which normally will supply the continuous dc load in the system. The battery capacity will be suitable for an emergency loading based on a failure of ac station service lasting 5 hours.

(iii) "Black Start" Capability

The Watana power plant will have a built-in capability of starting up a completely blacked-out power system in a very short time. Only a few basic requirements will have to be satisfied:

- Sufficient water will be available in the reservoir for the minimum generation required for "black start" operation;

- The governor oil system will have sufficient stored energy capable of operating the turbine wicket gates to full open position;

- The generators wille be equipped with static exciters capable of being flash-started from the station battery system

- Dc control power will be available for the startup circuits.

The above described emergency power requirements will not exceed about 200 kW for one unit and will be easily supplied from the emergency diesel generator. With the startup of a single unit, the complete power plant and switchyard auxiliary power will be immediately available, enabling all the units in the power plant to be started up sequentially within the hour.

(i) Grounding System

The power plant grounding system will consit of one mat under the power plant, one mat under the transformer gallery, risers, and connection ground wires. Grounding grids will also be included in each powerhouse floor. The power plant grounding system will be connected to the switchyard grounding system by three 500 MCM copper ground conductors to minimize the overall resistance to ground. The grounding system will be designed to provide a ground resistance of 1 ohm or lower. All exposed metal part and neutral connections of generators and transformers will be connected to the grounding system for the purpose of protecting personnel and equipment from injury or damage.

(j) Lighting System

The lighting system in the powerhouse will be supplied from 480/208-120 volts lighting transformers connected to the general ac auxiliary station service system. The lighting system will be all fluorescent and incandescent fixtures operating on 120 volts and all outdoor type high pressure sodium fixtures operating on 208 volts. The lighting level varies generally from 20 to 50 foot candles depending upon the powerhouse area; the higher levels will be at control areas. Adequate illumination will be provided on vertical switchboards with local lighting canopies.

An emergency lighting system will be provided at the power plant and at the control room at all critical operating locations with an illumination level of 2 foot candles. The emergency lighting system will operate from a separate 120 volt ac circuit which, by means of automatic transfer switches, will be automatically connected to the 125 volt dc system upon failure of the ac system.

(k) Communications

The power plant will be furnished with an internal communications system, including an automatic telephone switchboard system. A communication system will be provided at all powerhouse floors and galleries, transformer gallery, access tunnels and cable shafts, and structures at the intake, draft tube chamber, spillway, and dam.

The communications system for the central dispatch control system, telemetering, supervisory and protective relaying system is described in Section 15.

(l) Insulation Coordination and Lightning and Switching Surge Protection

The electrical insulation and protective devices will be selected and coordinated to provide a safe margin of insulation strength above the maximum abnormal voltages permitted during lightning, switching, and short-circuit surges. The 1300 kV basic insulation level (BIL) specified for the transformer and other BIL values stated for the electrical equipment and connections are tentative and are subject to detailed study in the design stage of the project.

In principle, lightning arresters will be mounted on or adjacent to all major electrical equipment having wound-type internal construction, and will be provided at the generator 15 kV terminals and the main transformer 345 kV terminals.

12.19 - Switchyard Structures and Equipment

TO FOLLOW

12.20 - Project Lands

Project lands acquired for the project will be the minimum necessary to construct access and site facilities, construct permanent facilities, to clear the reservoir, and to operate the project.

Appendix C contains land status background information relative to the Susitna Project, together with an inventory or private and public lands required for the project. A large amount of public land in the Watana area is managed by the Bureau of Land Management. There are large blocks of private Native Village Corporation Lands along the river. Other private holdings consist of widely scattered remote parcels. The state has selected much of the federal land in this area and is expected to receive a patent.

TABLE 12.1: WATANA PEAK WORK FORCE AND CAMP/VILLAGE DESIGN POPULATION

Calendar Year	Yearly Peak Force	Camp/Village Design
1985	910	1000
1986	1360	1500
1987	4005	4400
1988	5635	6200
1989	5635	6200
1990	5635	6200
1991	5635	6200
1992	4000	4000
1993	2000	2200
1994	1090	1200
1995	270	300

TABLE 12.2: ROCKFILL AND EARTH DAMS IN EXCESS OF 500 FEET

Dam	Country	Feet
Rogun	USSR	1,066
Nurek	USSR	1,040
Watana	USA	885
Tehri	India	856
Kishaw	India	830
Sulak	USSR	802
Mica	Canada	794
Patia	Colombia	787
Chicoasen	Mexico	787
Chivor	Colombia	778
Oroville	USA	771
Esmeradla	Colombia	754
Sayansk	USSR	738
Keban	Turkey	679
Altinkaya	Turkey	640
New Melones	USA	626
Don Pedro	USA	614
Swift	USA	610
Portage Mountain	Canada	600
New Bullards Bar	USA	590
Dartmouth	Australia	590
Okoy	Turkey	590
Ayvacik	Turkey	587
Takase	Japan	577
Hasan Ugurlu	Turkey	574
Nader Shah	Iran	574
Gura Apelor Retezat	Romania	568
Magarin	Jordan	561
Charvak	USSR	551
Boruca	Costa Rica	548
Kremasta	Greece	541
Trinity	USA	538
Thomson	Australia	530
Talbingo	Australia	530
Tokujama	Japan	528
LaGrande No. 2	Canada	525
Palo Quemado	S. America	525
Grand Maison	France	525
Sao Felix	Brazil	525
Fierze	Albania	519
Cougar	USA	519
Yacambu	Venezuela	519
Emborcacao	Brazil	519
Finstertal	Austria	519
Cumberland	Australia	510
Canales	Spain	510
Narmata	Japan	508
Goeschheneralp	Switzerland	508
Salrajina	Colombia	505
Gepätsch	Austria	503
Foz do Areia	Brazil	503
Tedorigawa	Japan	503
Carter	USA	500

TABLE 12.3: SUMMARY OF DESIGN DATA FOR LARGE EMBANKMENT DAMS IN SEISMICALLY ACTIVE AREAS

Dam	Height Feet	Freeboard	Crest Width Feet	Ratio of Core Width to Dam Height	Upstream Slope	Downstream Slope
Watana (U)	885	25*	35	0.50	2.4	2.0
Mica (C)	794	26	111	0.45	2.25	2.0
Chicoosen (M)	787	33	82	0.42	2.2	2.0
Oroville (U)	771	22	80	0.34	2.6	2.0
Don Pedro (U)	614	-	-	-	2.4	2.1
Ayvacik (T)	587	17	50	0.34	2.5	1.8
Takase (J)	577	17	46	0.40	2.6	2.1
Tedorigawa (J)	503	13	40	0.31	2.6	1.85
Netzahualcoyotl (M)	453	18	50	0.43	2.0	2.0
Iwaya (J)	413	62	33	0.33	2.5	2.0
Kazurya (J)	413	-	39	-	2.6	1.8
Narakura (J)	410	16	39	0.56	2.7	2.7
Pyramid (J)	400	-	36	-	2.5	2.0
Tamahara (J)	380	13	39	0.43	2.7	2.2
Seto (J)	364	20	36	0.29	2.5	2.0

* Watana freeboard - normal maximum operations level to nominal crest (additional height allowed for seismic slumping)

TABLE 12.4: DAMS IN SEISMIC AREAS

PROJECT			DIMENSIONS				IMPERVIOUS CORE							FILTERS		FOUNDATION		
Name	Country	Seismic Activity	Height (ft)	Crest Length (ft)	Free-board (ft)	Crest Width (ft)	Core Type	Core Width at Base (ft)	Ratio Core Width to Dam Height	Slopes of Core Zone	Unified Classi-fication	Liquid Limit	Plastic Limit	U/S Thick-ness (ft)	D/S Thick-ness (ft)	Type	Treat-ment	Under Shells
Watana	USA	H	885		25	35	CV	440	0.50	0.25:1	SM	23	8	60	60	R	CG	R/UD
Mica	Canada	L	794	2,600	26	111	S	360	0.45	–	–	–	–	–	–	R	CG	A/UD
Chicoasen	Mexico	H	787	1,640	33	82	CV	330	0.42	0.15:1	CL	40	20	25	25	CB	CG	A/UD
Oroville	USA	L-M	771	5,600	22	80	S	263	0.34	–	GC	–	–	–	–	–	CB	–
Ayvacik	Turkey	M-H	587	1,400	17	50	C	197	0.34	–	CL	–	–	50	50	R	CB	R/UD
Takase	Japan	M-H	577	1,200	17	46	CV	230	0.40	0.15:1	–	–	–	–	50	R	–	A/UD
Falo Quemado	S. America	H	525	1,215	26	40	CV	295	0.56	0.25:1	ML	33	7.5	13	13	R	CG	R/UD
Tedorigawa	Japan	M-H	503	1,380	13	40	CV	157	0.31	0.15:1	–	–	–	26	26	R	–	A/UD
El Infiernilla	Mexico	H	486	1,100	25	40	CV	164	0.34	0.15:1	CL	49	25	8	8	R	CG	A/UD
Tarbela	Pakistan	M-H	469	9,000	18	40	S	262	0.56	–	GW/SM	–	–	–	–	A	–	A/UD
Netzahualcoyotl	Mexico	H	453	1,570	18	50	CV	197	0.43	0.15:1	ML/MH	50	20	–	13	R	CG	A/U
Mangla	Pakistan	H	453	3,400	32	41	S	230	0.51	–	CL	–	–	–	13	R	CG	A/UD
Derbendi Khan	Iraq	M	443	1,460	33	56	CV	330	0.74	0.3:1	CH/CL	50	26	20	30	R	–	R/UD
Tsengwen	Taiwan	H	436	1,440	33	33	CV	410	0.94	0.4:1	SM/GM	22	8	–	–	R	CG	R/UD
Pueblo Viejo	C. America	H	436	820	49	43	CV	157	0.36	0.15:1	CL	41	19	23	23	R	CG	R/UD
Beas	India	M-H	435	6,400	30	45	CV	131	0.30	0.1:1	CL	30	12	20	20	R	–	A/UD
Alicura	Argentina	H	426	2,620	16	39	CV	275	0.65	0.3:1	CL	35	15	10	10	R	–	A/UD
Ramganga	India	M-H	413	–	22	39	CV	197	0.48	0.2:1	CL	–	–	98	79	R	–	A/UD
Iwaya	Japan	H	413	1,200	62	33	C	138	0.33	–	–	–	–	33	65	R	–	R/UD
Narakura	Japan	M-H	410	820	16	39	CV	230	0.56	0.2:1	–	–	–	20	49	R	–	A/UD
Shimokotori	Japan	M-H	390	915	13	36	C	151	0.39	–	–	–	–	39	39	–	–	R/UD
Bao	C. America	H	388	1,312	24	26	CV	184	0.47	0.2:1	CL	40	20	20	20	R	–	CB & CG
Tamahara	Japan	H	380	2,000	13	39	CV	164	0.43	–	–	–	–	79	79	R	CG	R/UD
Seto	Japan	H	364	1,120	13	36	C	105	0.29	–	–	–	–	39	39	R	–	R/UD
Guri	Venezuela	L	361	1,970	20	36	CV	180	0.50	0.2:1	ML	50	15	5	6.5	R	CG	R/UD

Legend

Earthquake:

H – High
M – Medium
L – Low

Impervious Core:

CV – Central vertical
C – Central very slightly sloped
S – Sloping

Foundation:

R – Rock
A – Alluvium
D – Downstream
U – Upstream
CG – Consolidation grouting
CB – Concrete block over rock

TABLE 12.5: GENERALIZED SURFICIAL STRATIGRAPHIC COLUMN AREA "D" AND RELICT CHANNEL

Column	Unit	Estimated Thickness	Description
-	Surficial	0-5'	Boulders, organic silts and sands.
C	Outwash	0-18' 12' average	Silty sand with some gravel and cobbles occasionally. Usually brown although becomes gray in limited areas. Thickest in northern portions of area, thickening southward, often absent near Susitna River.
D	Alluvium & Fluvial Deposits	0-15'	Sand with some silt, occasional gravel. Generally brown, found only along course of limited drainage channels formed in outwash "E". Generally sorted.
E and F	Outwash	0-35' 15' average	Sand, silt, gravel and cobbles, partly sorted, with fragments sub-angular to rounded. Silt and sand lenses often present. Brown to gray brown with a cobble/boulder zone often present at the base of Unit "F". Contact between "E" and "F" is often poorly defined.
G	Till/Waterlain Till	2-50' 12' average	Clayey, silty sand, usually gray, often plastic. Contains cobbles and gravel in many areas. Occasionally present as a lacustrine deposit showing laminations and/or varves. Generally a till deposited through or near standing water.
H	Alluvium	0-40'	Sand, silt, gravel, partly to well sorted. Often absent between Units "I" and "G". Unit represents period of melting producing alluvium/outwash between these deposits. Appears as narrow bands representing channel fillings. Thickest in western portion of the area.
I and J	Till	>10' to 65' 20' average	Poorly sorted sand, silt, gravel and cobbles, occasionally with clay. Generally gray to gray brown. Continuity uncertain due to lack of information at depth. Silt or sand layer 2 inches - 6 inches thick often found in center of Unit "I". Base unit on top of bedrock, except in buried channel. Contact between "I" and "J" often poorly defined.
-	Alluvium	to 160'	Sand, gravel, cobbles, boulders, few fines, permeable. Found only in bottom of buried channel. Top at 292 feet extending to rock at 454 feet.

Note: Letters used to define units are arbitrary and were used for correlation purposes. Two letters may define parts of the same unit.

TABLE 12.6: RING FOLLOWER GATES

PROJECT	LOCATION	SIZE (IN.)	HEAD (FT)	YEAR OF INITIAL OPERATION
(1) New Melones	California	96	607	1979
(2) New Melones	California	72	591	1979
(3) Portage Mountain	Canada	84	550	1967
(4) Hungry House	Montana	96	495	1952
(5) Yellowtail Dam	Montana	84	470	1967
(6) Trinity Dam	California	84	450	1962
(7) Grand Coulee	Washington	102	354*	1940
(8) Glen Canyon	Colorado	96	337	1965
(9) Green Mountain	Colorado	102	261	1943

*Maximum static head; maximum operating head - 250 feet.

TABLE 12.7: PRELIMINARY UNIT DATA

1 - GENERAL DATA

- Number of Units	6
- Nominal Unit Output	170 MW
- Headwater Levels:	
-- normal maximum	El. 2185
-- minimum	El. 2045
- Tailwater Levels:	
-- minimum	El. 1452
-- normal	El. 1459
-- maximum	El. 1465

2 - TURBINE DATA

- Type	Vertical Francis
- Rated Net Head	680 feet
- Maximum Head	724 feet
- Minimum Head	584 feet
- Full Gate Output:	
-- at rated head	250,000 hp
-- at maximum head	275,000 hp
-- at minimum head	200,000 hp
- Best Gate Output	85 percent full
- Full Gate Discharge at Rated Head	3560 cfs
- Speed	225 rpm
- Specific Speed	32.4
- Runner Discharge Diameter	132 in
- Runaway Speed	385 rpm
- Centerline Distributor	El. 1422
- Cavitation Coefficient (sigma)	0.081

3 - GENERATOR DATA

- Type	Vertical Modified Umbrella
- Rated Output	190 MVA
- Power Factor	0.90
- Voltage	15 kV
- Synchronous Speed	225 rpm
- Inertia Constant (H)*	3.5 MW/sec/MVA
- Flywheel Effect (WR^2)*	52×10^6 lb-ft^2
- Heaviest Lift	770,000 lb

*Including turbine

TABLE 12.8: ASSUMED PROPERTIES FOR STATIC ANALYSES OF WATANA DAM

Material		K	Kur	n	Rf	Kb	m	C			Ko
CORE:											
-- Soft(1)	140	200	300	.8	.6	60	.8	0	35	0	.43
-- Stiff(2)	140	700	800	.35	.8	280	.2	0	35	0	.43
TRANSITION(3)	145	1300	1500	.4	.72	900	.22	0	35	6	.43
SHELLS (4)	145	1800	2000	.4	.67	1300	.16	0	35	6	.43

where:

= Unit weight, pcf
K = Modulus number, ksf
Kur = Elastic unloading modulus number, ksf
n = Modulus exponent
Rf = Failure ratio
Kb = Bulk modulus number, ksf
m = Bulk modulus exponent
C = Cohesion, psf
= Friction angle, degrees
= Decrease in friction angle perlog cycle increase in $_3$, degrees
Ko = Earth pressure coefficient

Note: Values taken from Duncan et al., 1980, "Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stress and Movements in Soil Masses," Report No. UCB/GT/80-01, University of California, Berkeley.

(1) Mica Creek Dam Core, 2 percent wet of optimum
(2) Mica Creek Dam Core, 2 percent dry of optimum
(3) Oroville Dam silty sandy gravel
(4) Oroville Dam Shell - Amphibolite gravel

TABLE 12.9: WATANA DAM - CREST ELEVATION AND FREEBOARD

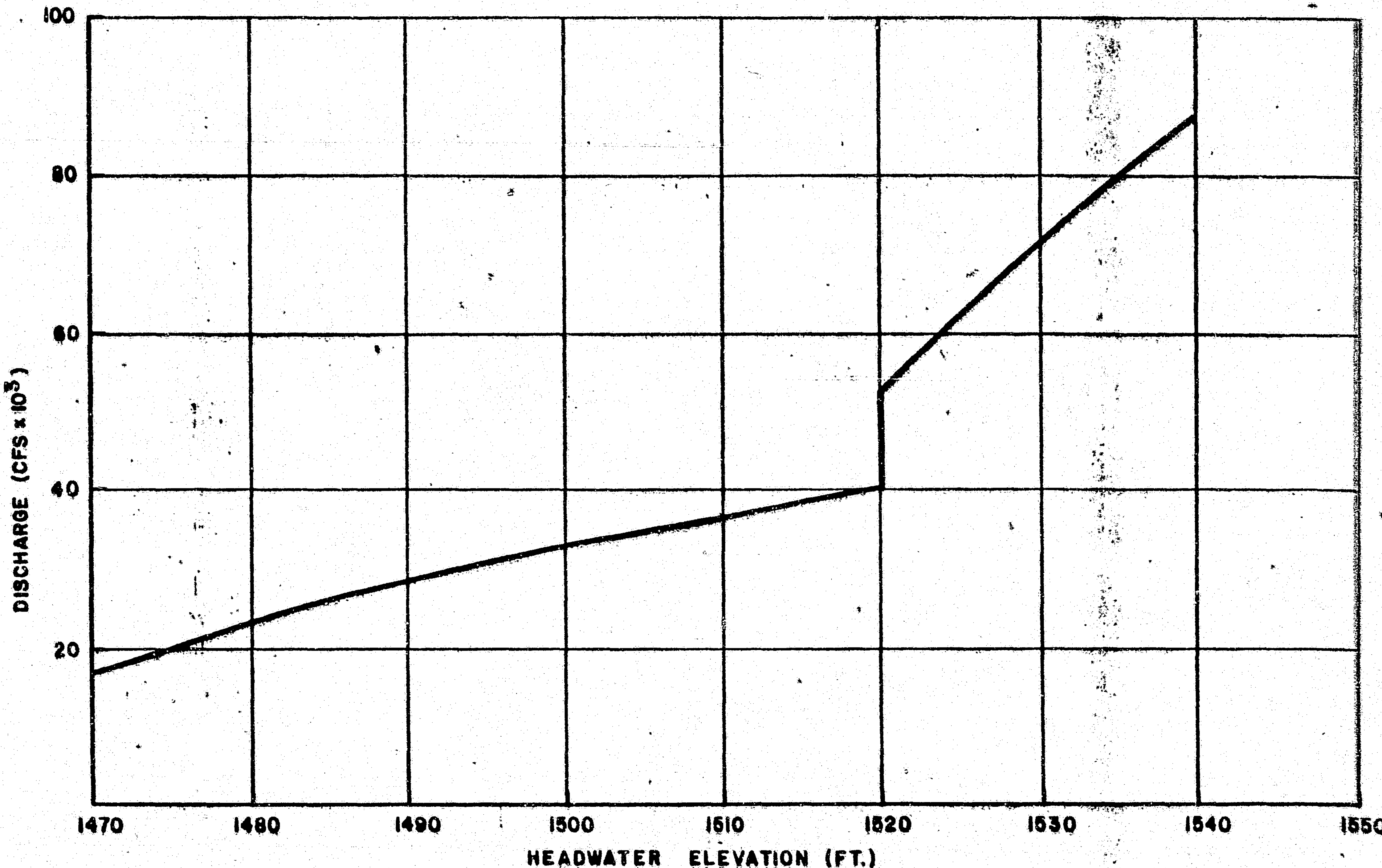
River Inflow	1 in 50 Year Storm	1 in 10,000 Year Storm	Probable Maximum Flood
Normal maximum reservoir elevation	2185	2185	2185
Storm surcharge	6	8	17
Still water elevation	2191	2193	2202
Wave runup allowance	6)	6	NIL
Dry freeboard allowance	3)		
Elevation top of core	2200	2199	2202
Roadway over core	3	3	3
Minimum crest elevation	2203	2202	2205

Governing elevation for crest of main dam	2205
Highest still water level to be 2 feet above fuse plug pilot channel	
Sill of pilot channel in fuse plug	2200

Note: The above elevations do not include allowances for static settlement and seismic slumping.

TABLE 12.10: RECENT HIGH HEAD FRANCIS TURBINES

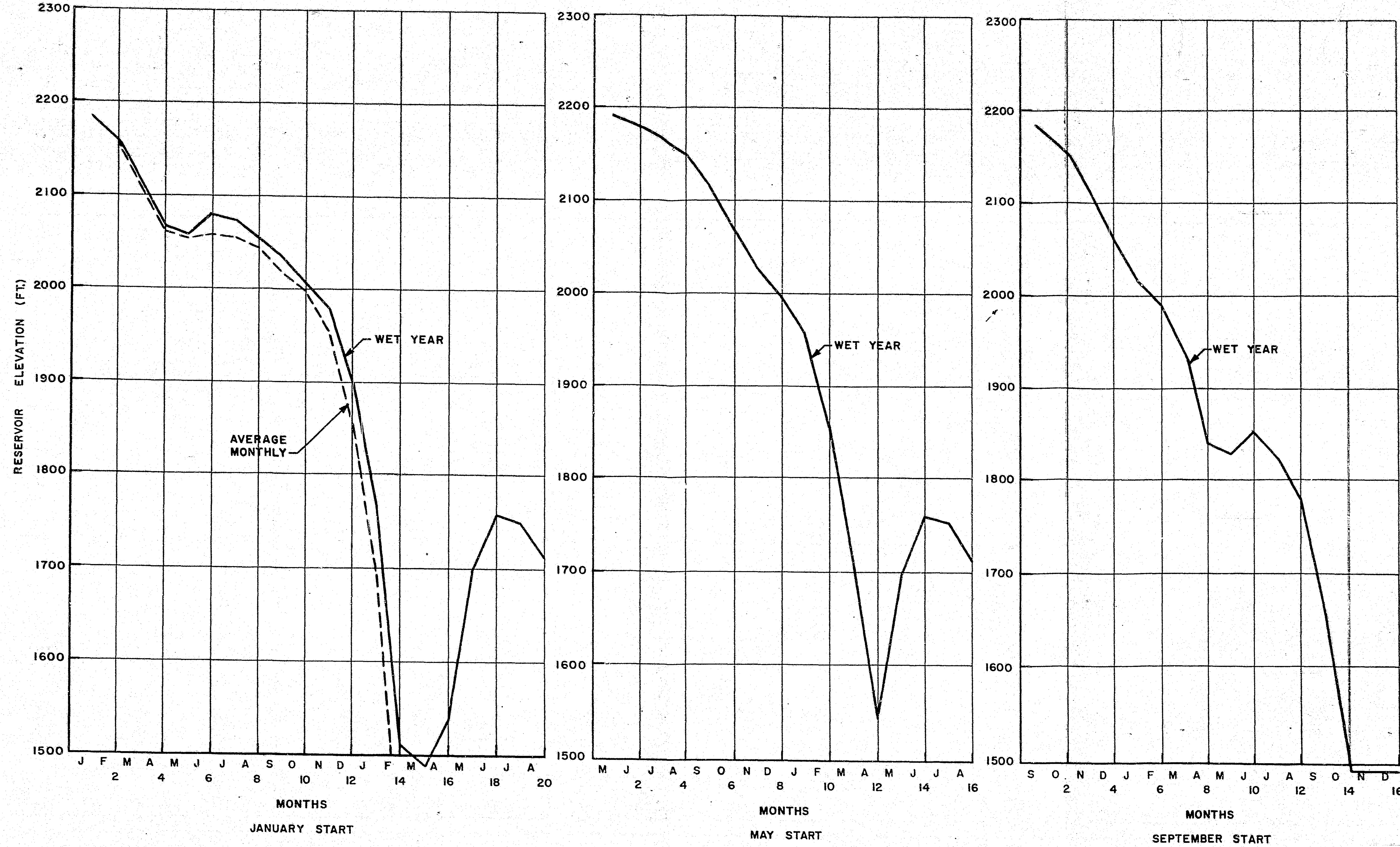
Number	Plant	Year of Order	Head (ft)	Output (hp)	Speed (rpm)	Specific Speed (Ns)
1	Albi	1972	1141	49,100	750	25.1
2	Alto Anchicaya	1970	1312	158,000	450	22.6
3	Big Creek	1976	737	50,600	450	26.4
4	El Chocon	1971	192	274,000	88.3	64.7
5	Hendrik-Verwoerd	1972	225	137,000	136.4	57.9
6	Estreito	1970	208	239,000	112.5	69.6
7	Grand Coulee III	1973	285	808,000	72	55.3
8	Grand Coulee IV	1973	285	938,000	85.7	70.9
9	Grimsel II	1974	1502	142,000	750	30.2
10	Harspranget IV	1974	338	245,000	166.7	56.9
11	Hermillon	1971	535	82,300	333	37.1
12	Inga II	1972	205	239,000	107.1	67.5
13	Kargamakis	1970	443	185,000	214	45.3
14	Langsan	1972	590	70,500	428	39.1
15	La Suassaz	1970	679	109,000	333	31.7
16	LG-2	1975	450	454,000	133	43.2
17	Libby	1970	300	165,000	128.6	41.8
18	Loentsch	1970	1178	54,200	750	25.3
19	Magisano	1971	1215	52,800	750	24.0
20	Malpaso	1974	313	293,000	128.6	52.9
21	Marimbondo	1972	236	248,000	100	53.8
22	Mica	1975	560	600,000	128.6	36.4
23	Mitta	1971	666	132,000	333.3	49.6
24	New Melones	1974	460	205,000	171.4	36.4
25	Nurek	1970	754	416,000	200	32.6
26	Oldan	1972	827	92,400	500	34.3
27	Passo Fundo	1972	853	151,000	360	25.3
28	Porjus	1971	195	323,000	83.3	65.0
29	Peace River	1971	450	410,000	150	36.9
30	Reza Shah-Kabir	1970	541	373,000	166.7	39.0
31	Ritsem	1973	476	442,000	166.7	49.8
32	Revelstoke	1977	427	664,000	112.5	47.2
33	Salas	1970	862	73,000	500	28.9
34	Salto Osorio	1972	236	212,000	120	59.7
35	Sarelli	1973	1149	65,700	750	28.7
36	Sirikit	1975	277	202,000	125	49.7
37	Sodusu II	1973	1246	55,200	600	19.0
38	Tumut 3	1981	530	379,000	187.5	45.3
39	Ust-Ilim	1972	296	328,000	125	58.3
40	Verbano II	1970	932	84,000	500	28.1
41	Waldeck II	1970	1104	295,000	375	32.0
42	Yarnvagsforsen	1973	278.8	72,900	214.3	50.8



WATANA DIVERSION
TOTAL FACILITY RATING CURVE

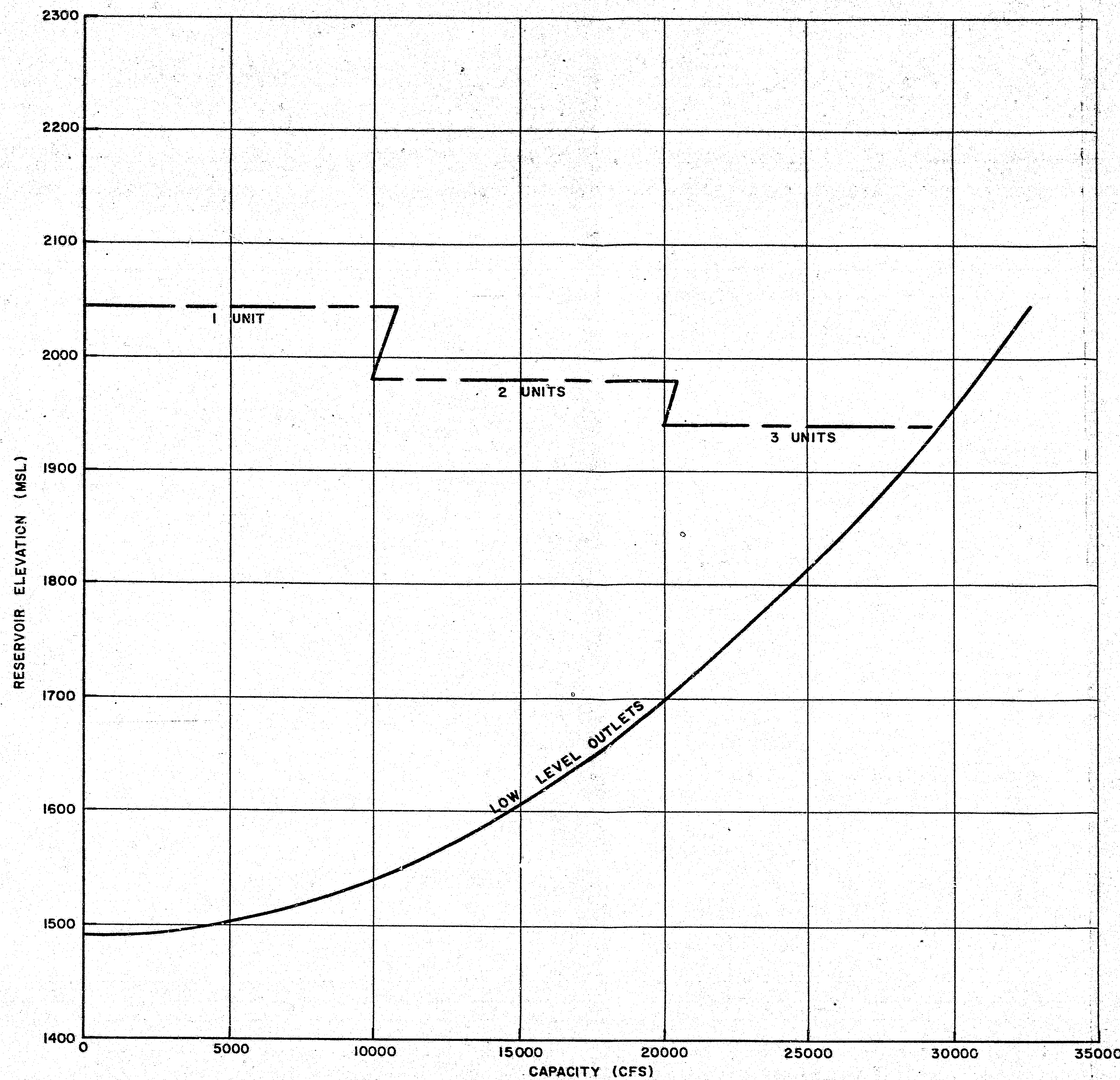
FIGURE 12.1

ACRES



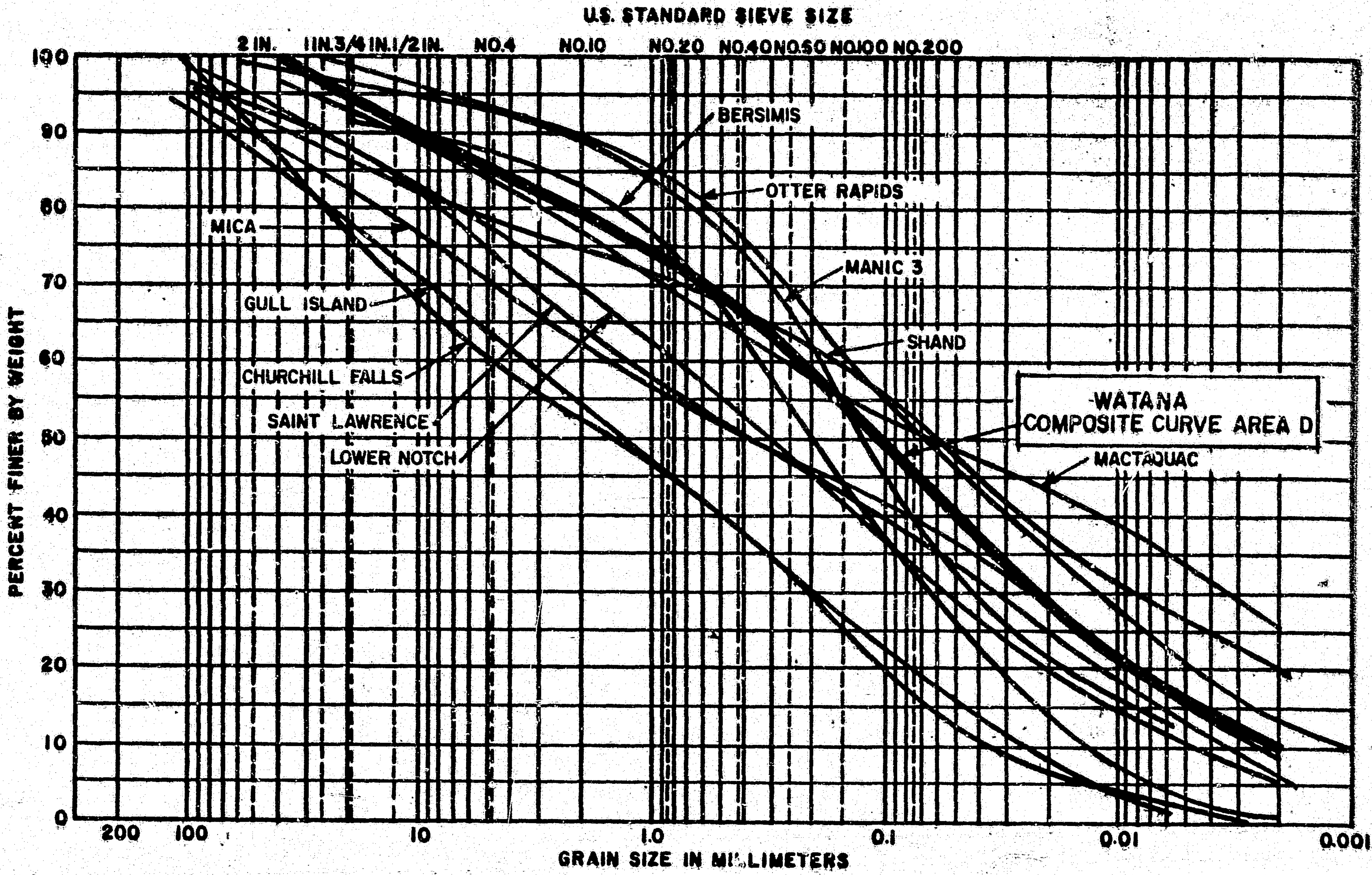
WATANA RESERVOIR
EMERGENCY DRAWDOWN

FIGURE 12.3



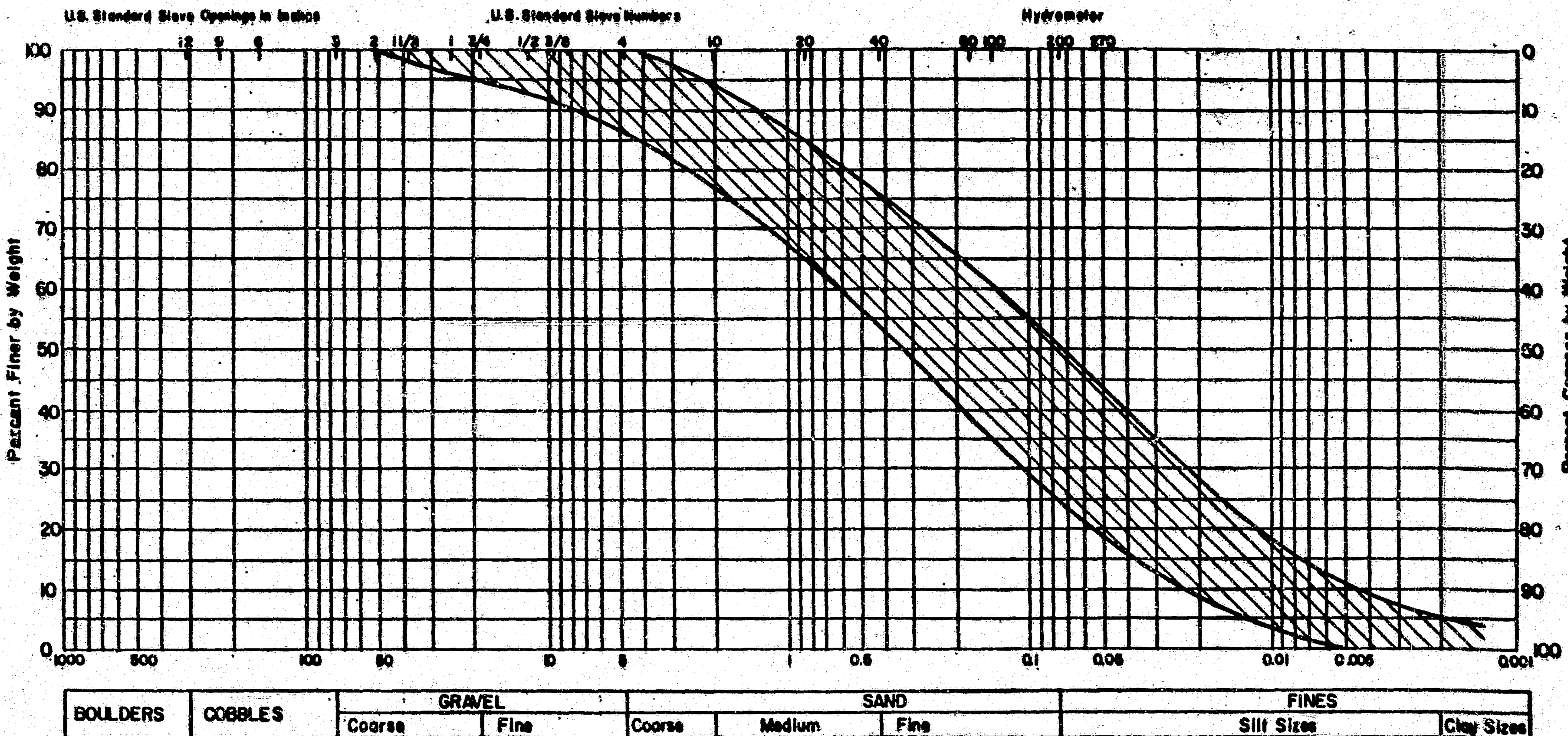
WATANA - LOW LEVEL OUTLETS - RATING CURVE

FIGURE 12.4



WATANA
COMPARISON OF GRAIN SIZE
CURVES FOR VARIOUS CORE MATERIALS

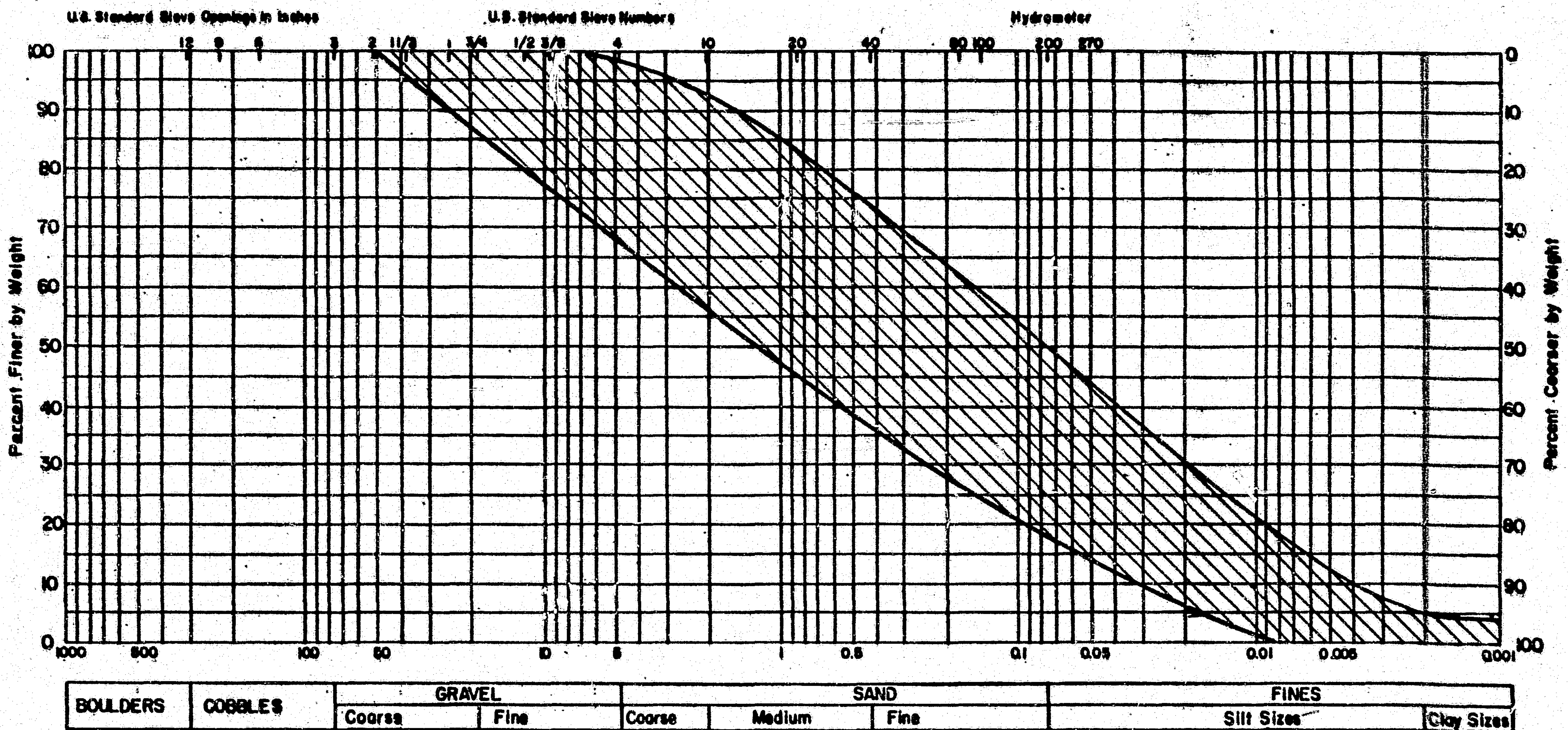
FIGURE 12.5



	RANGE	AVERAGE
MOISTURE CONTENT	8-21 %	12.8 %
LIQUID LIMIT	0-17	12.1
PLASTICITY INDEX	NP-2	NP

WATANA-BORROW AREA D
GRAIN SIZE CURVES — UNITS C, D + C/D

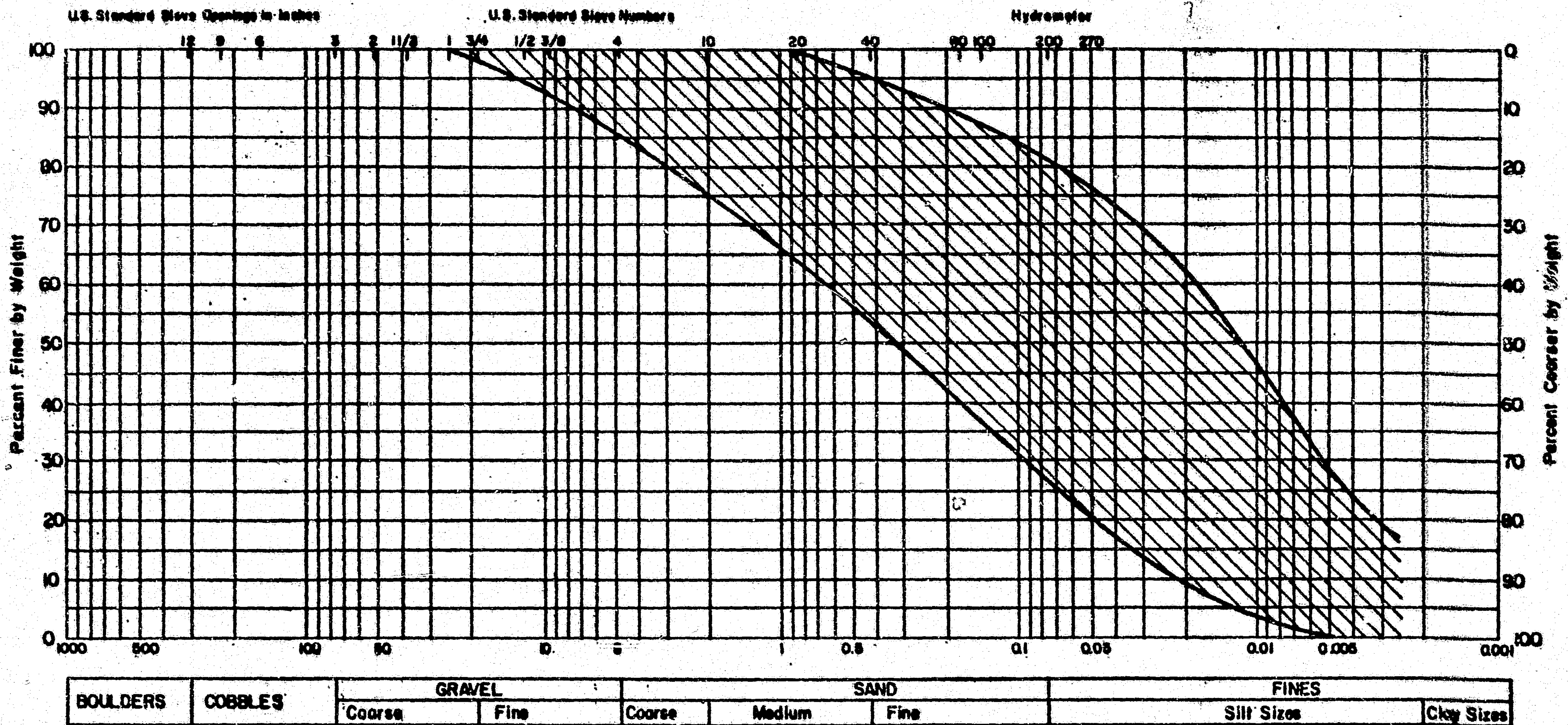
FIGURE 12.6



	RANGE	AVERAGE
MOISTURE CONTENT	6.6-13 %	10.2 %
LIQUID LIMIT	0-17	12.8
PLASTICITY INDEX	NP-2	NP

WATANA-BORROW AREA D
GRAIN SIZE CURVES — UNITS E,F + E/F

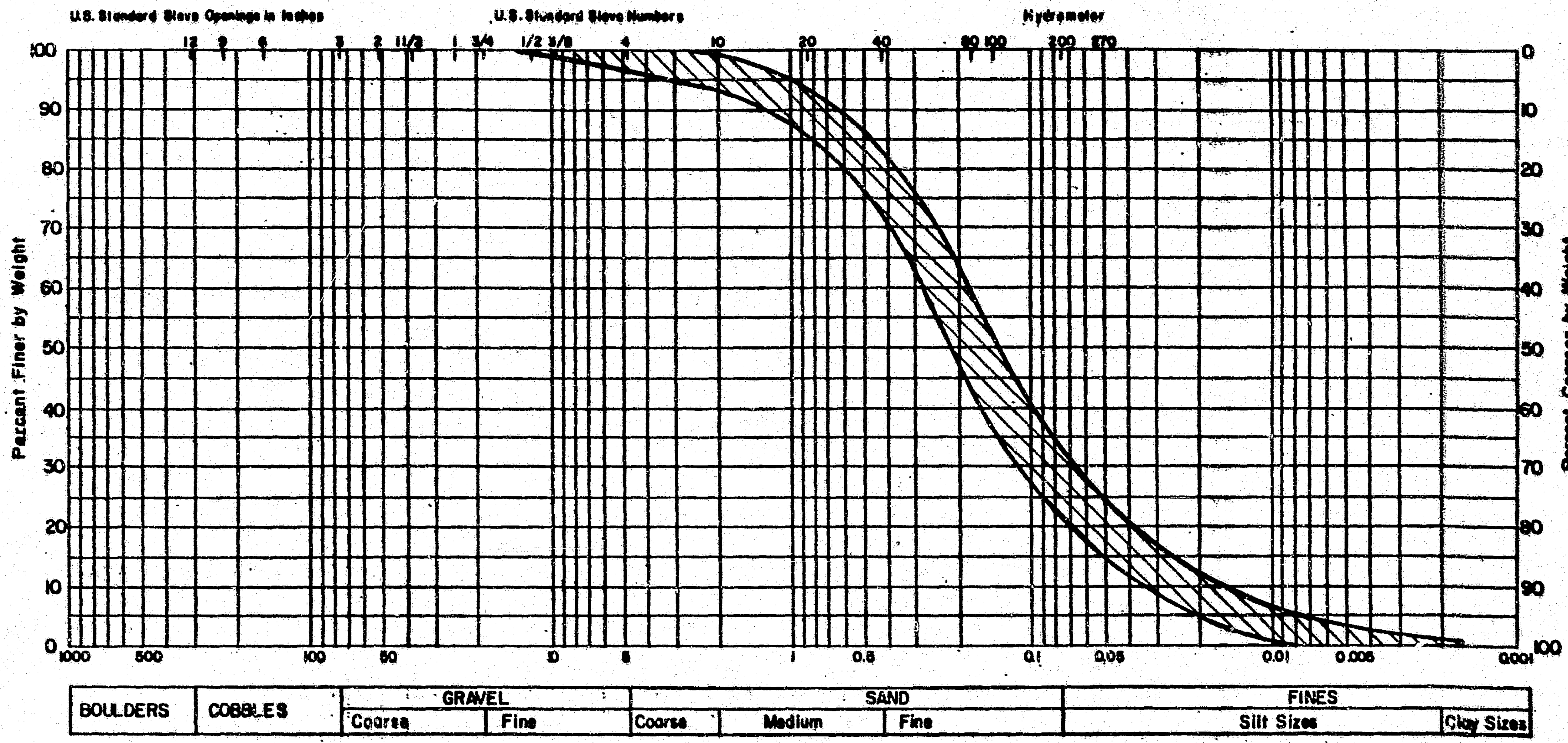
FIGURE 12.7



	RANGE	AVERAGE
MOISTURE CONTENT	6-40 %	20.2 %
LIQUID LIMIT	17-35	26
PLASTICITY INDEX	NP-15	10

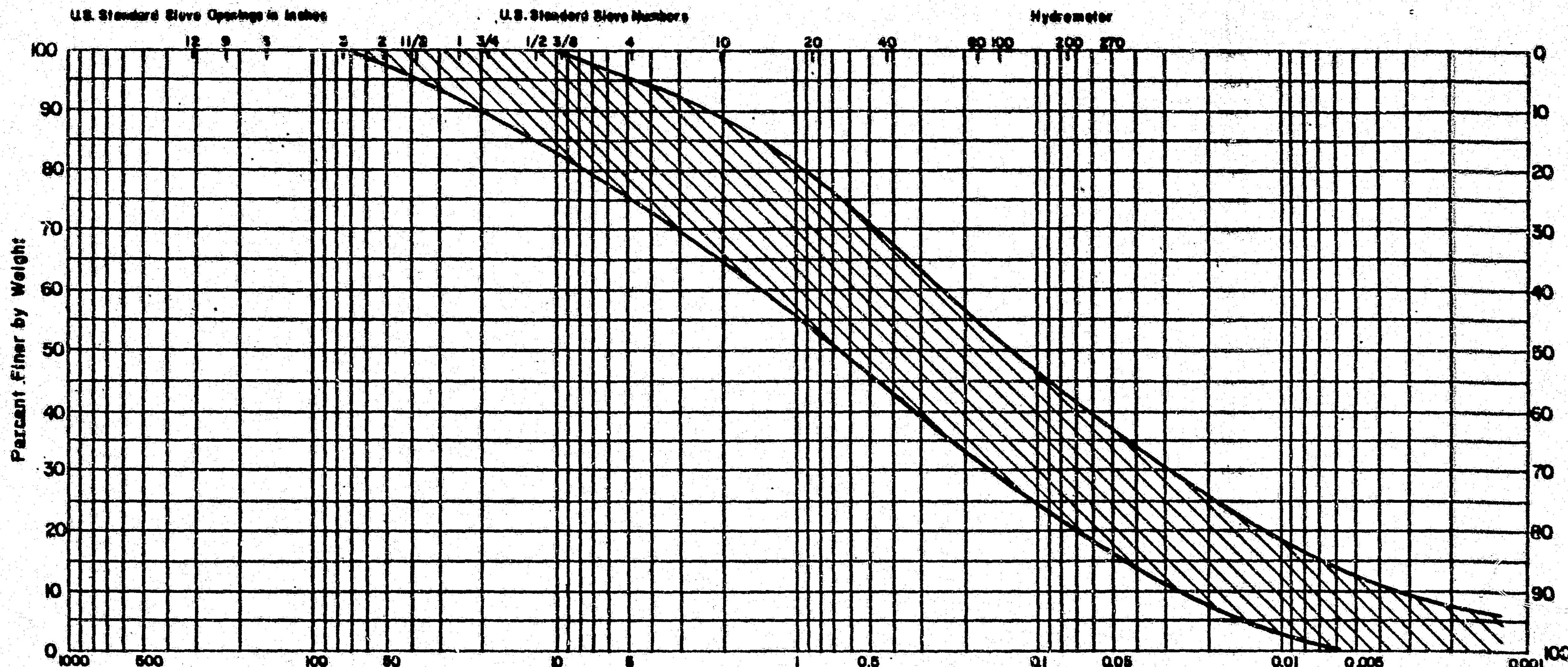
WATANA-BORROW AREA D
GRAIN SIZE CURVES — UNITS G+F/G

FIGURE 12.8



WATANA-BORROW AREA D
GRAIN SIZE CURVES – UNIT H

FIGURE 12.9

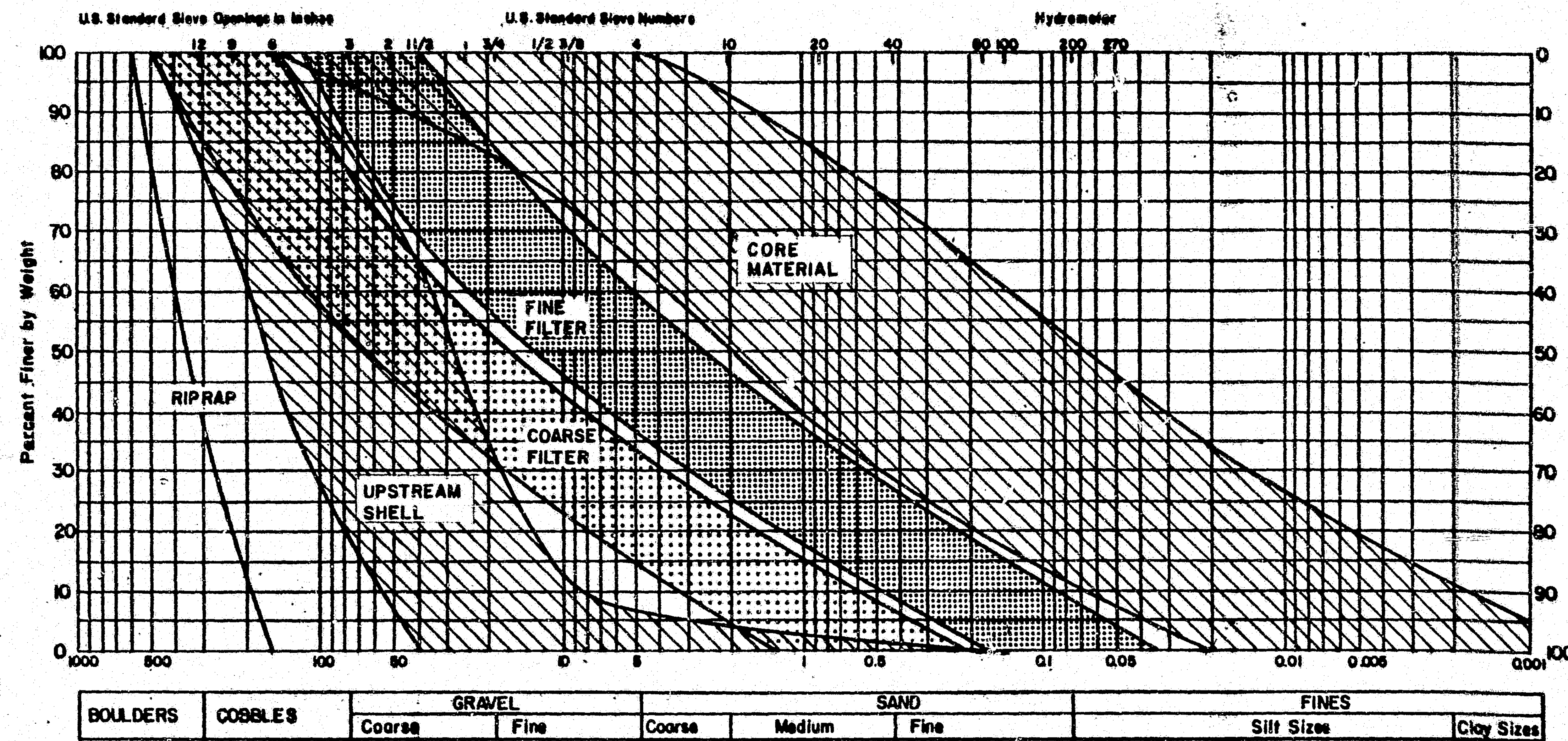


BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt Sizes	Clay Sizes

	RANGE	AVERAGE
MOISTURE CONTENT	9-13 %	10.3 %
LIQUID LIMIT	19.2-39	29.1
PLASTICITY INDEX	0.9-14	7.5

WATANA-BORROW AREA D
GRAIN SIZE CURVES – UNITS I, J + I/J

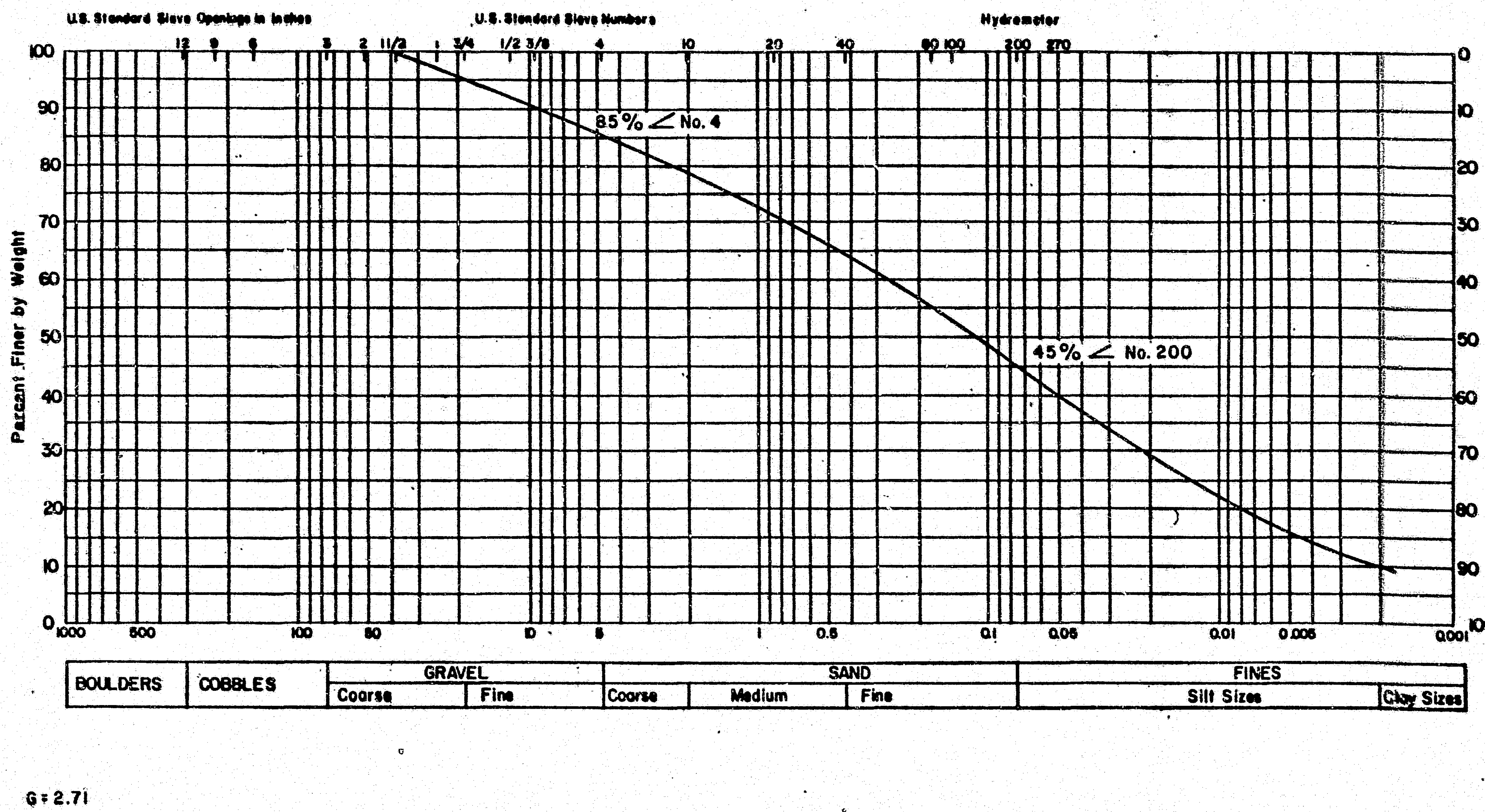
FIGURE 12.10



WATANA
REQUIRED GRAIN SIZE CURVES
MAIN DAM

ACRES

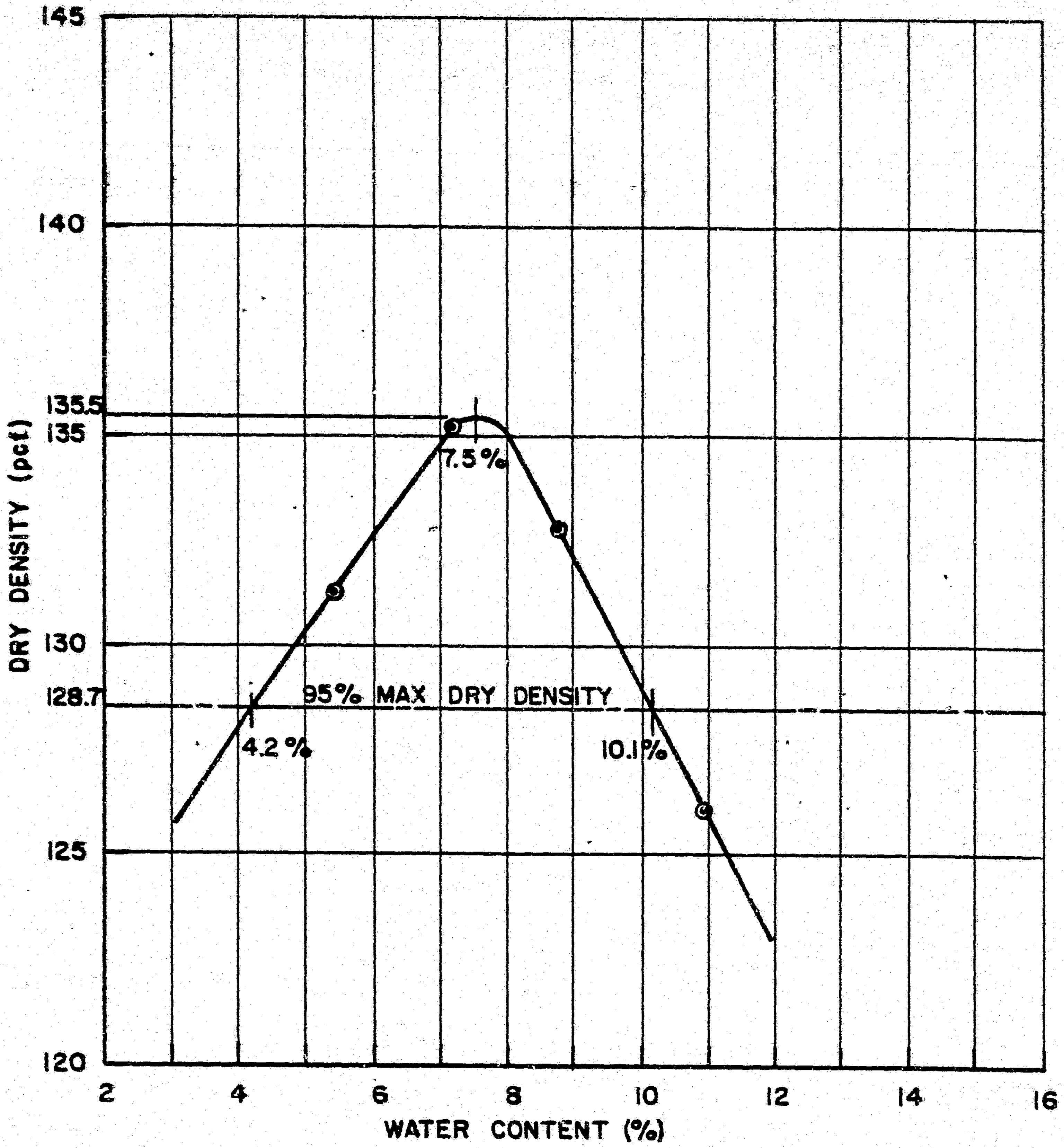
FIGURE 12.11



WATANA

COMPOSITE GRAIN SIZE CURVE – BORROW AREA D

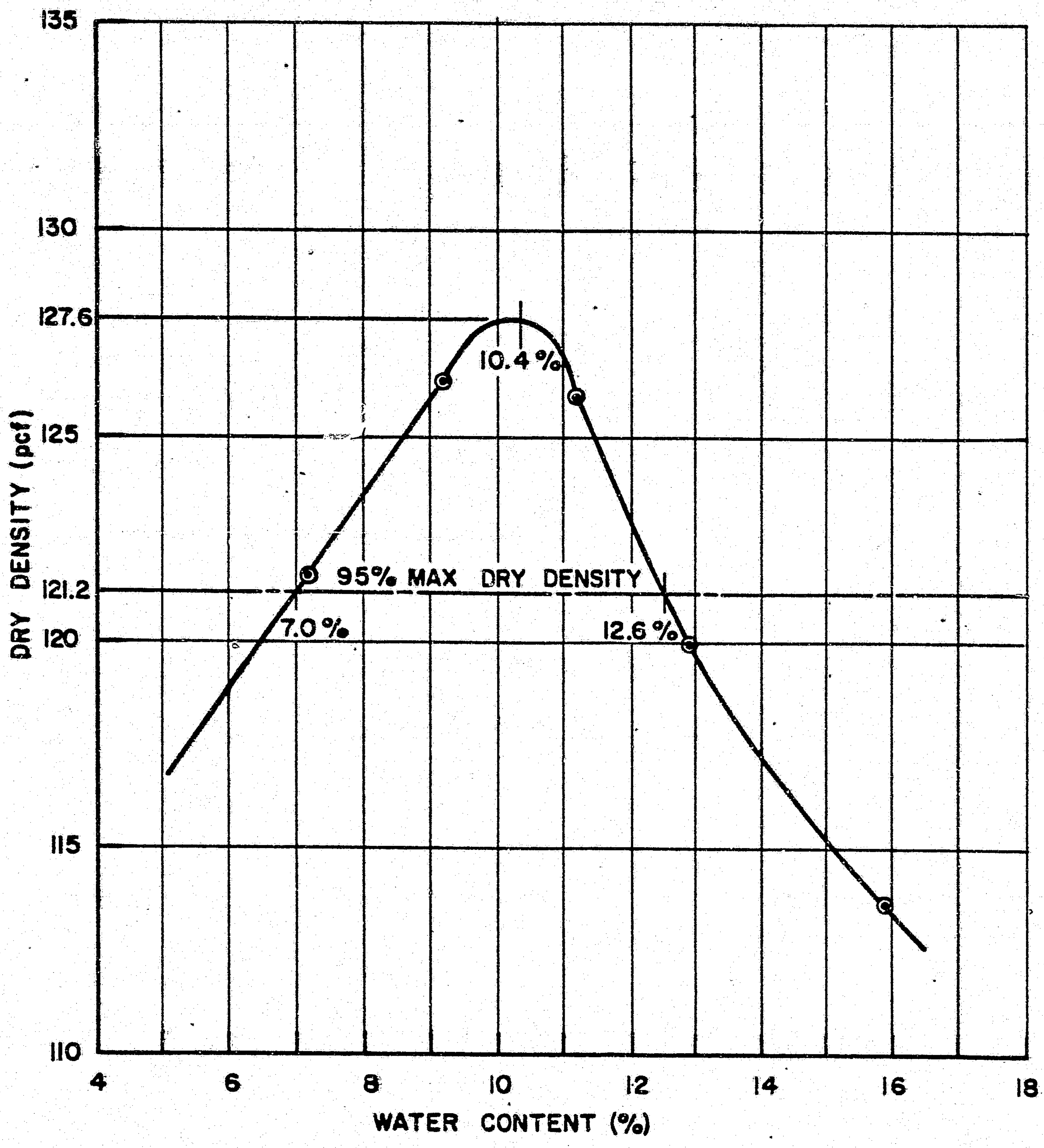
FIGURE 12.12



NOTE:
MATERIAL PASSING 3/4" SIEVE

WATANA
MODIFIED PROCTOR COMPACTION
COMPOSITE SAMPLE
BORROW AREA D

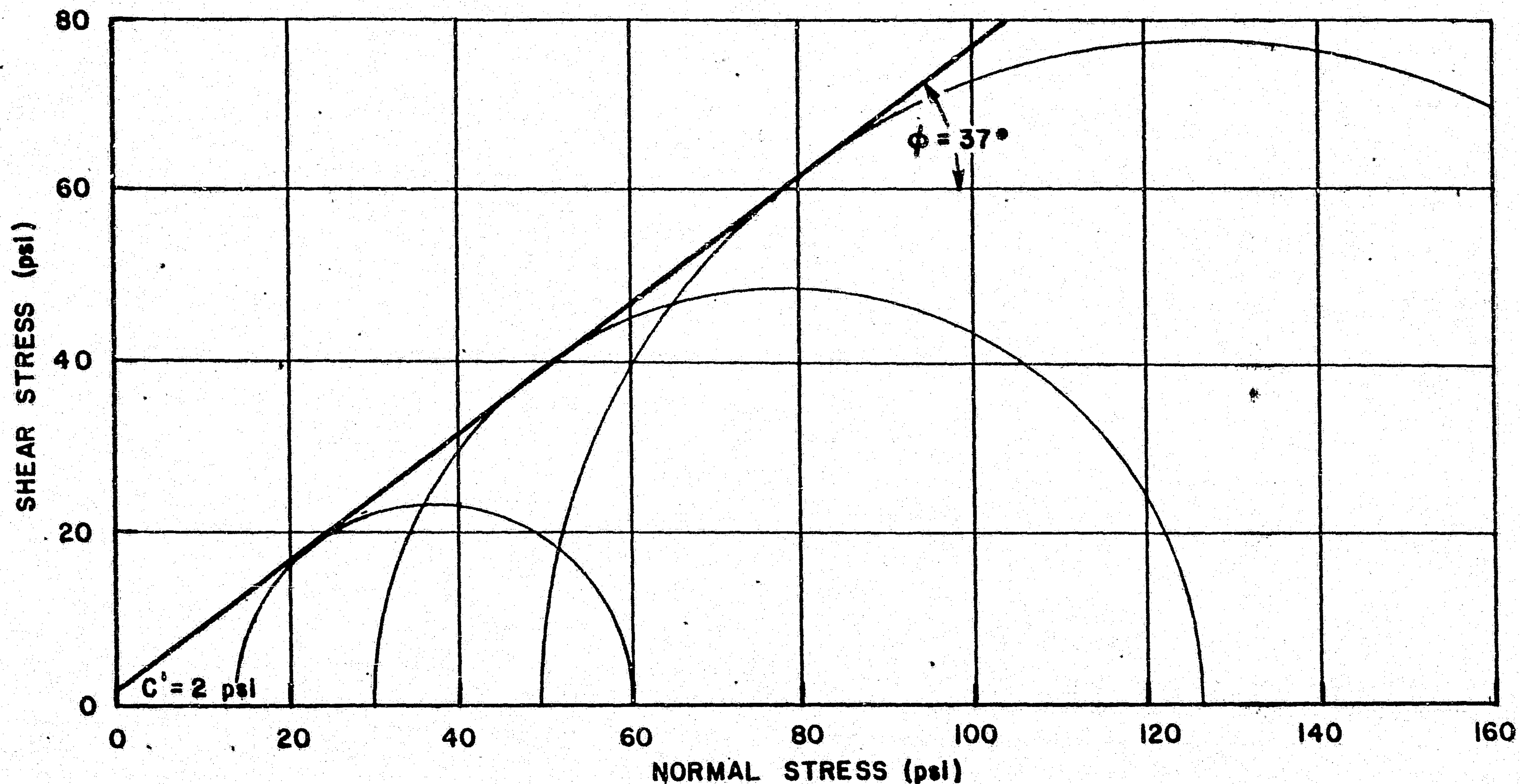
FIGURE 12.13



NOTE:
MATERIAL PASSING No. 4 SIEVE

WATANA
STANDARD PROCTOR COMPACTION
COMPOSITE SAMPLE
BORROW AREA D

FIGURE 12.14



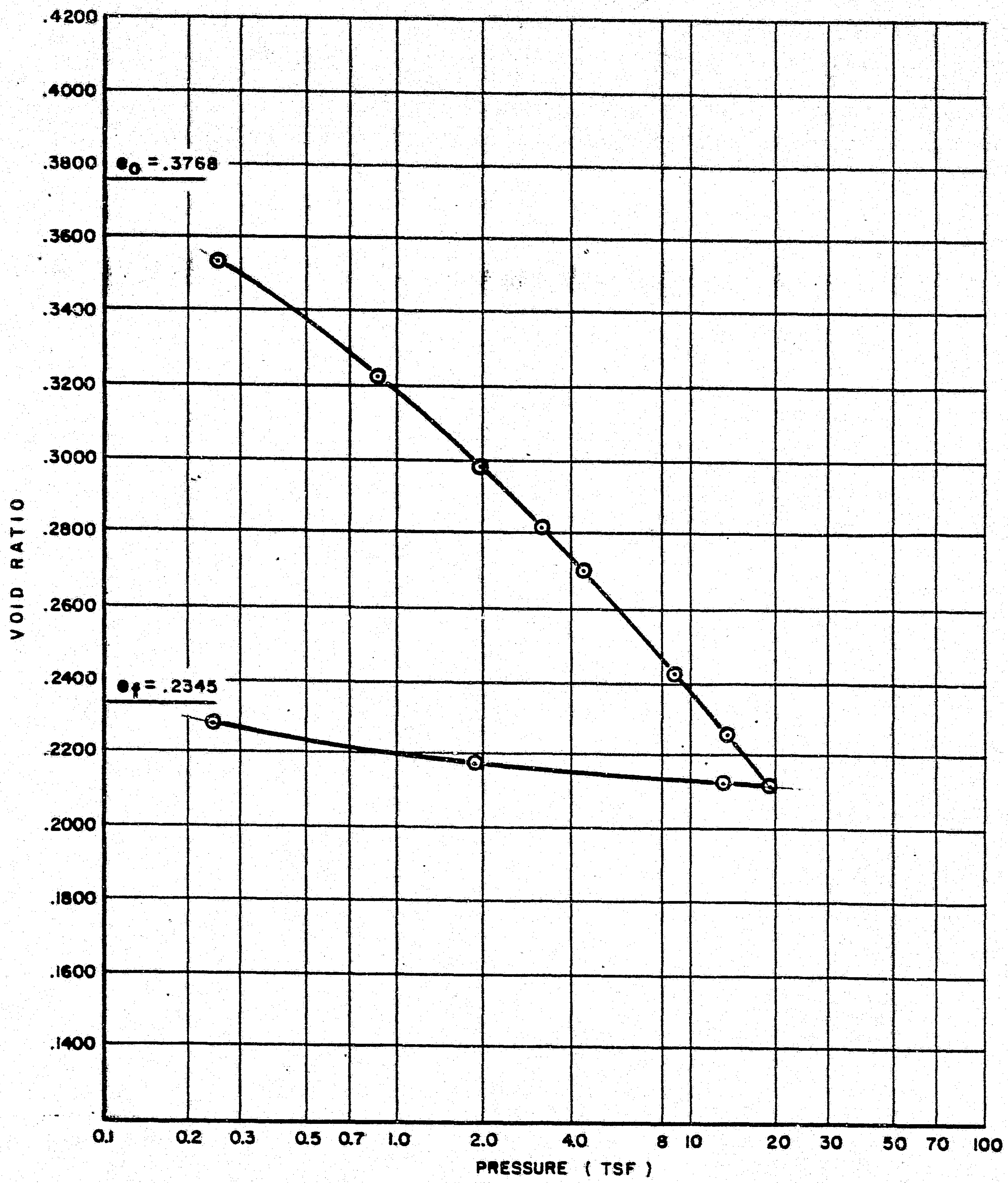
NOTE:

EFFECTIVE STRESS-CONSOLIDATED UNDRAINED SHEAR TEST, 4 INCH DIAMETER SAMPLES, OPT + 2% AT 95% MODIFIED PROCTOR COMPACTION.

WATANA
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
COMPOSITE SAMPLE — BORROW AREA D

FIGURE 12.15

ACRES

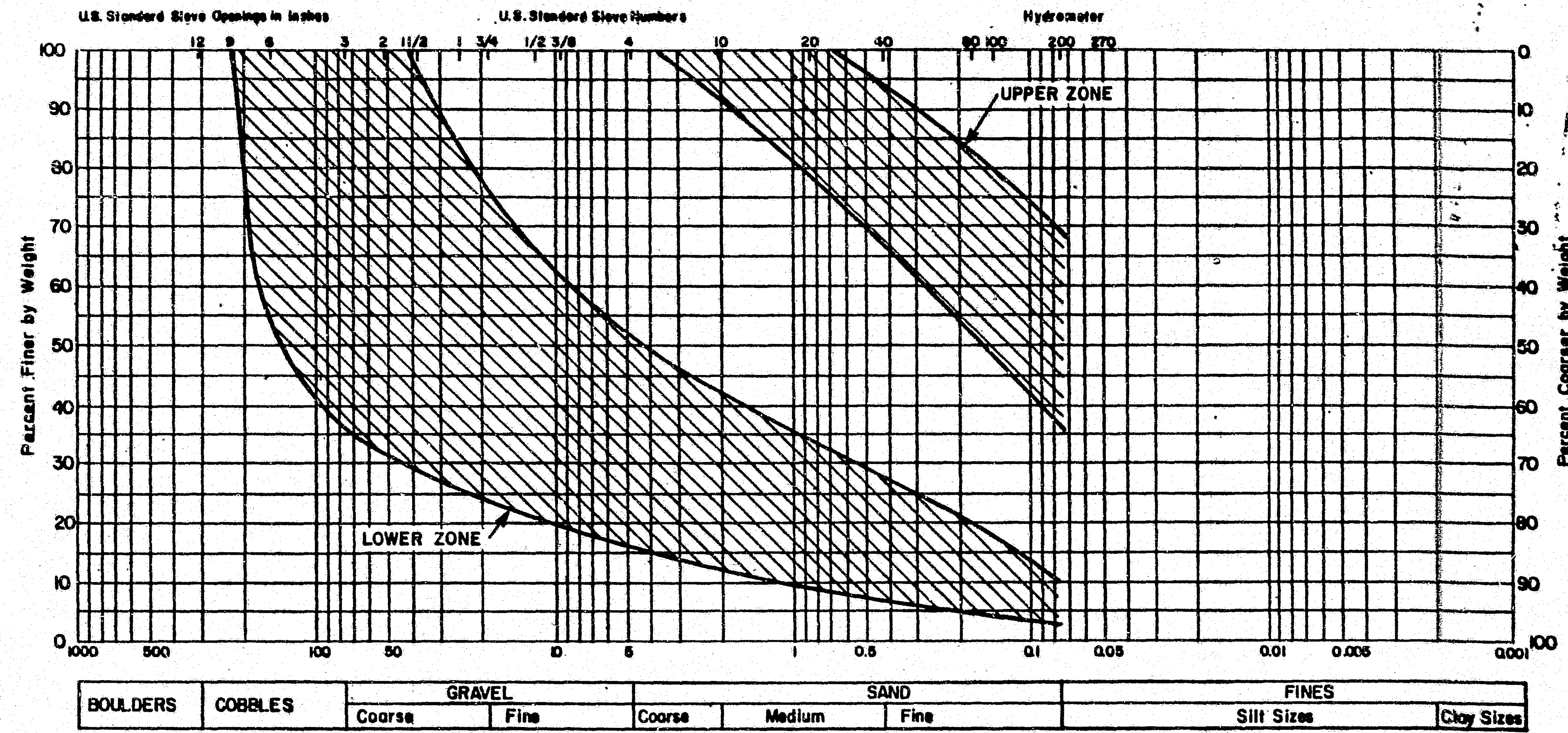


WATANA

CONSOLIDATION TEST- BORROW AREA D

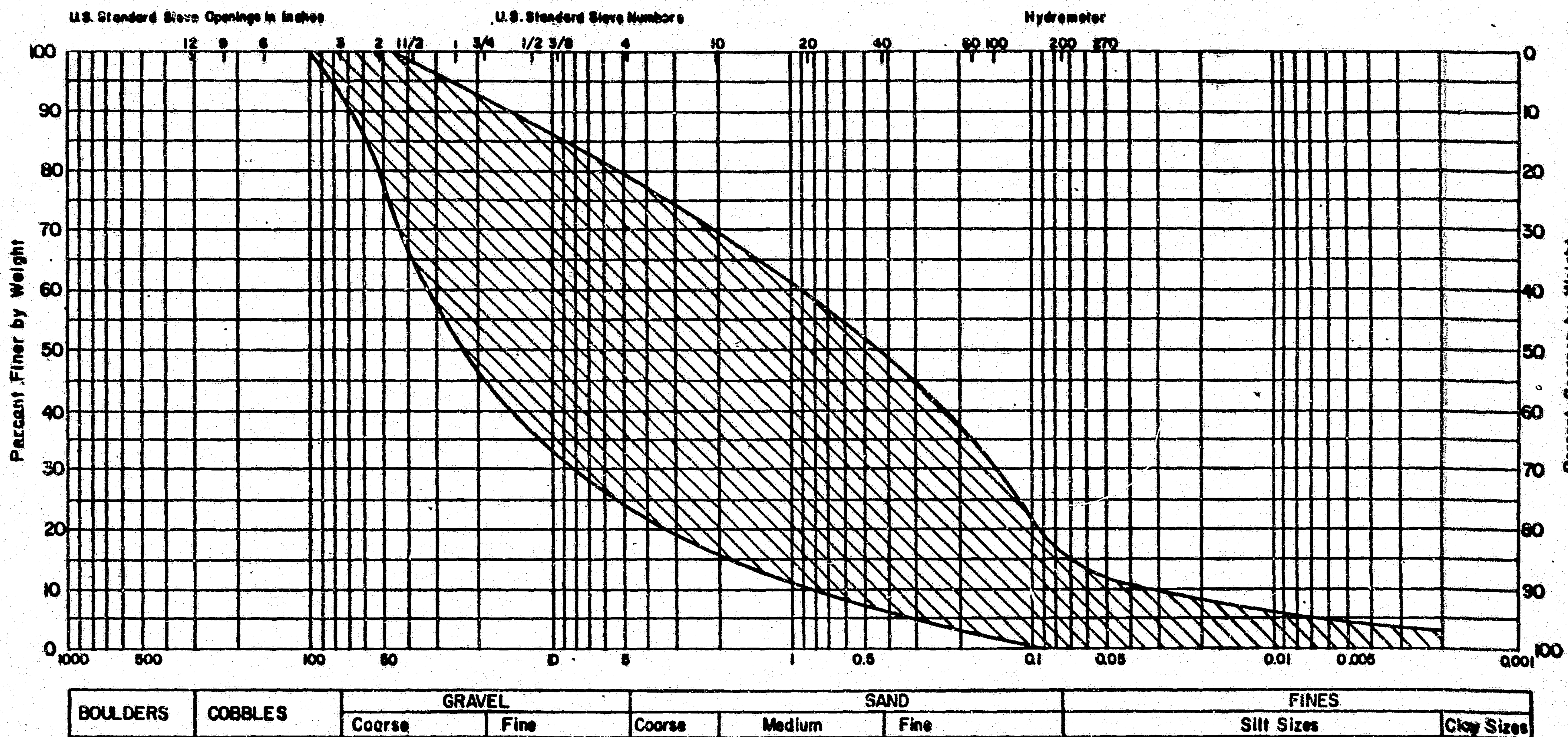
STANDARD PROCTOR COMPACTION—OPT. +2 %

FIGURE 12.16



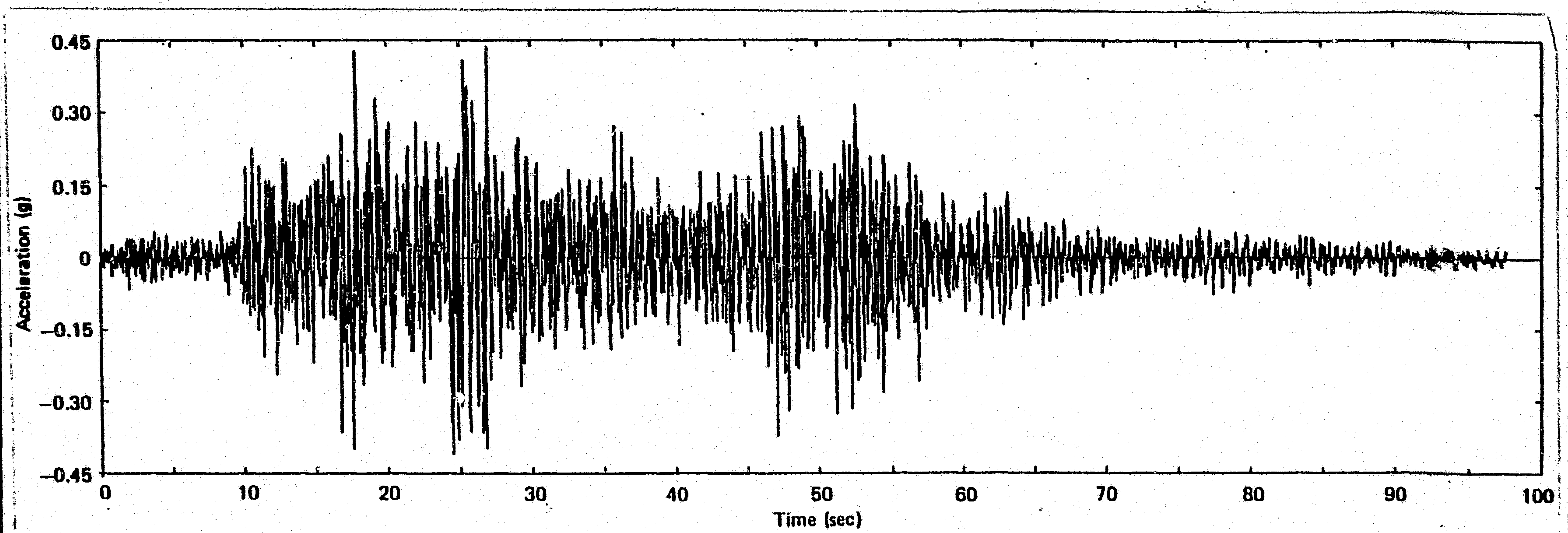
WATANA
GRAIN SIZE CURVES - BORROW AREA E

FIGURE 12.17



WATANA
GRAIN SIZE CURVES - BORROW AREA I

FIGURE 12.18

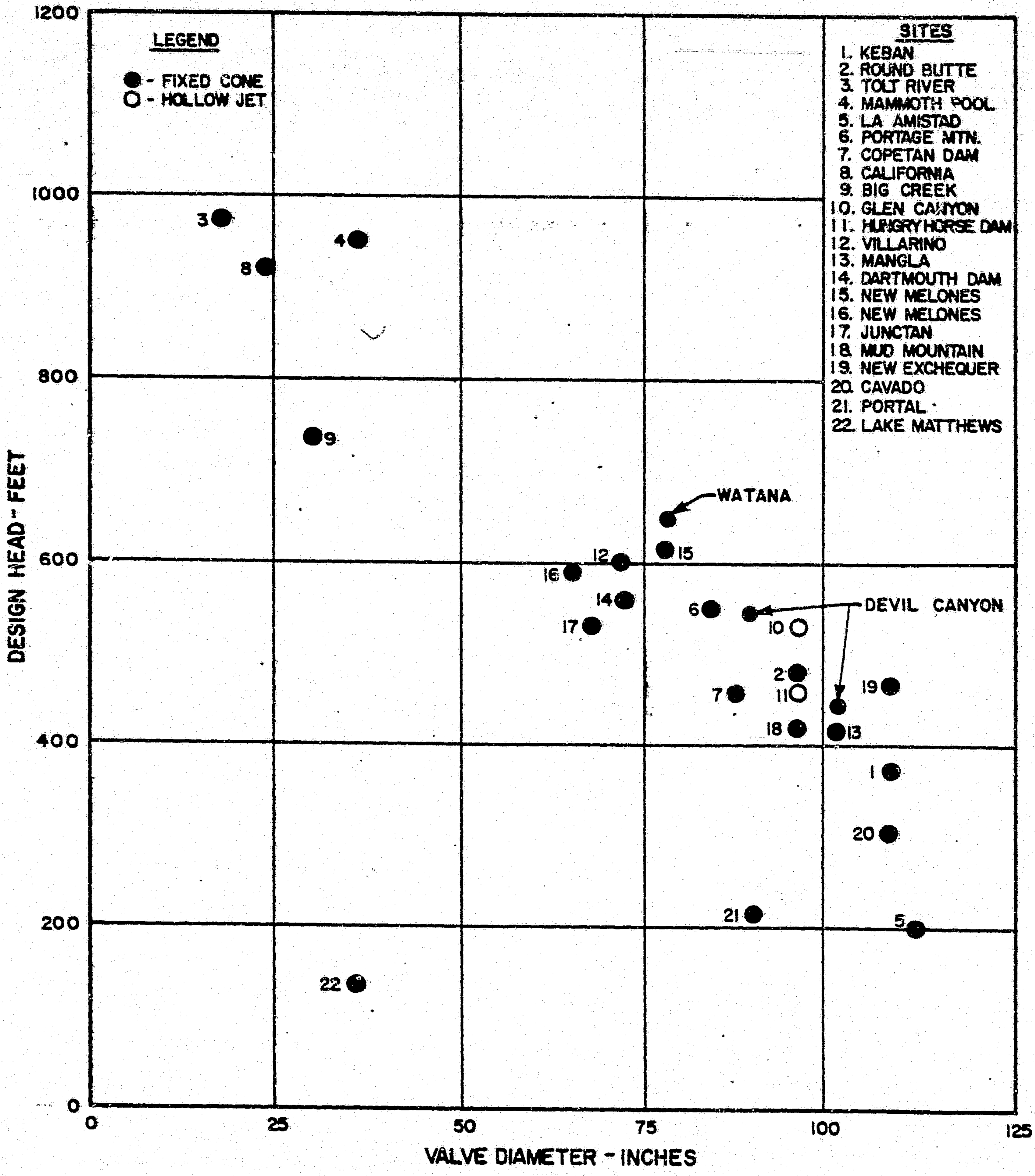


WATANA MAIN DAM
EARTHQUAKE TIME HISTORY

DRAFT

ACRES

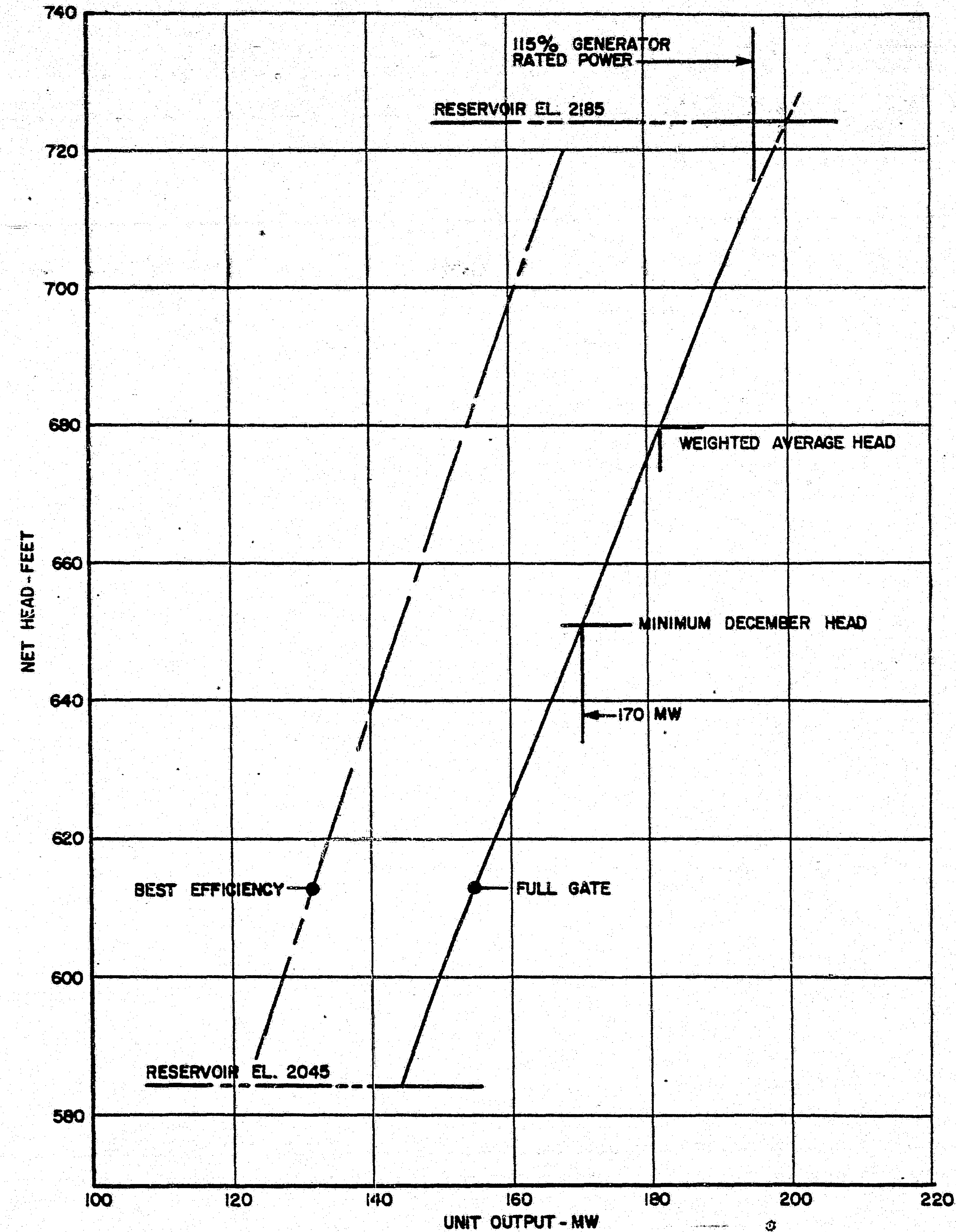
FIGURE 12.19



FREE DISCHARGE VALVE
EXPERIENCE PLOT

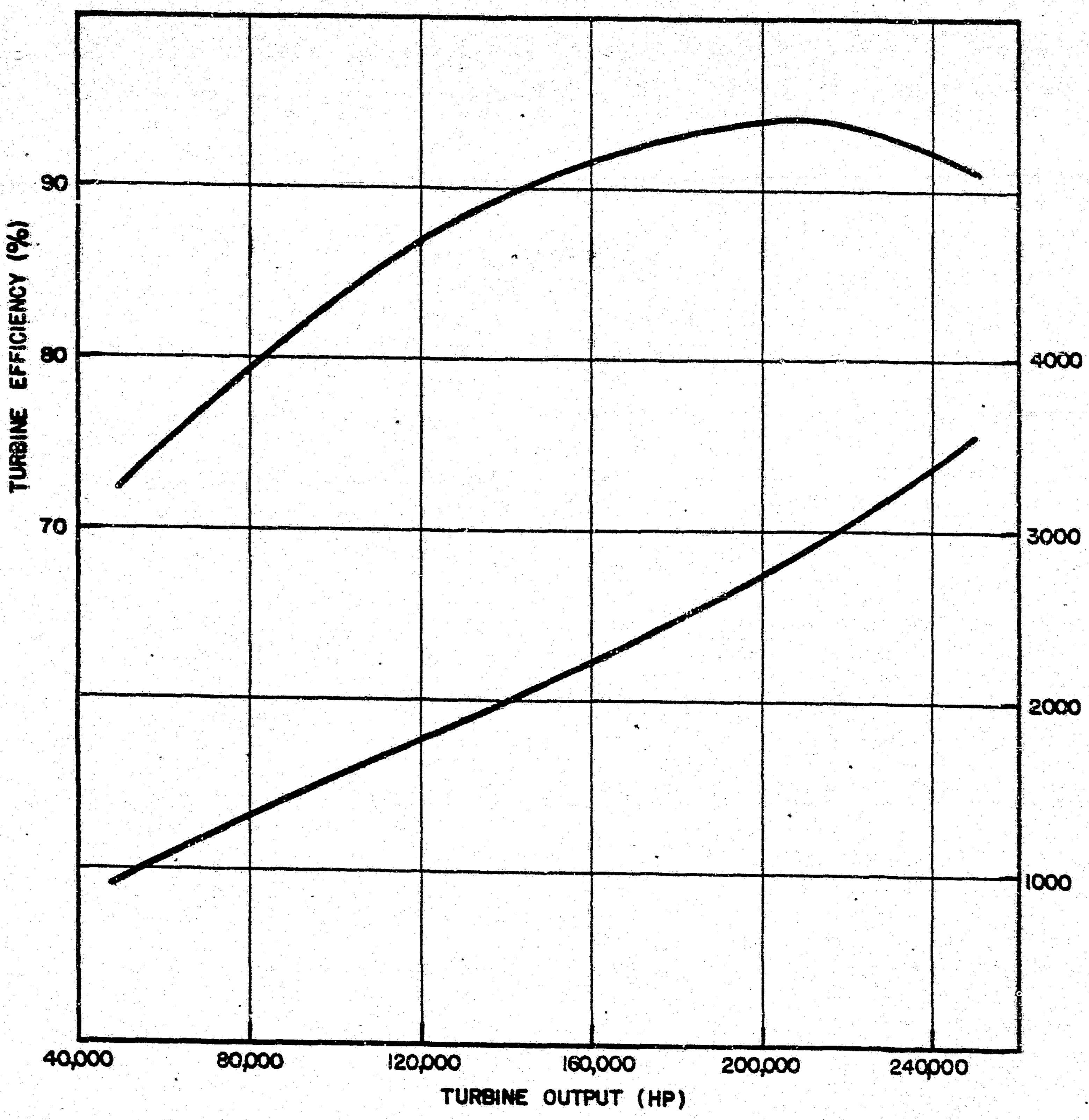
FIGURE 12.20

ACRES



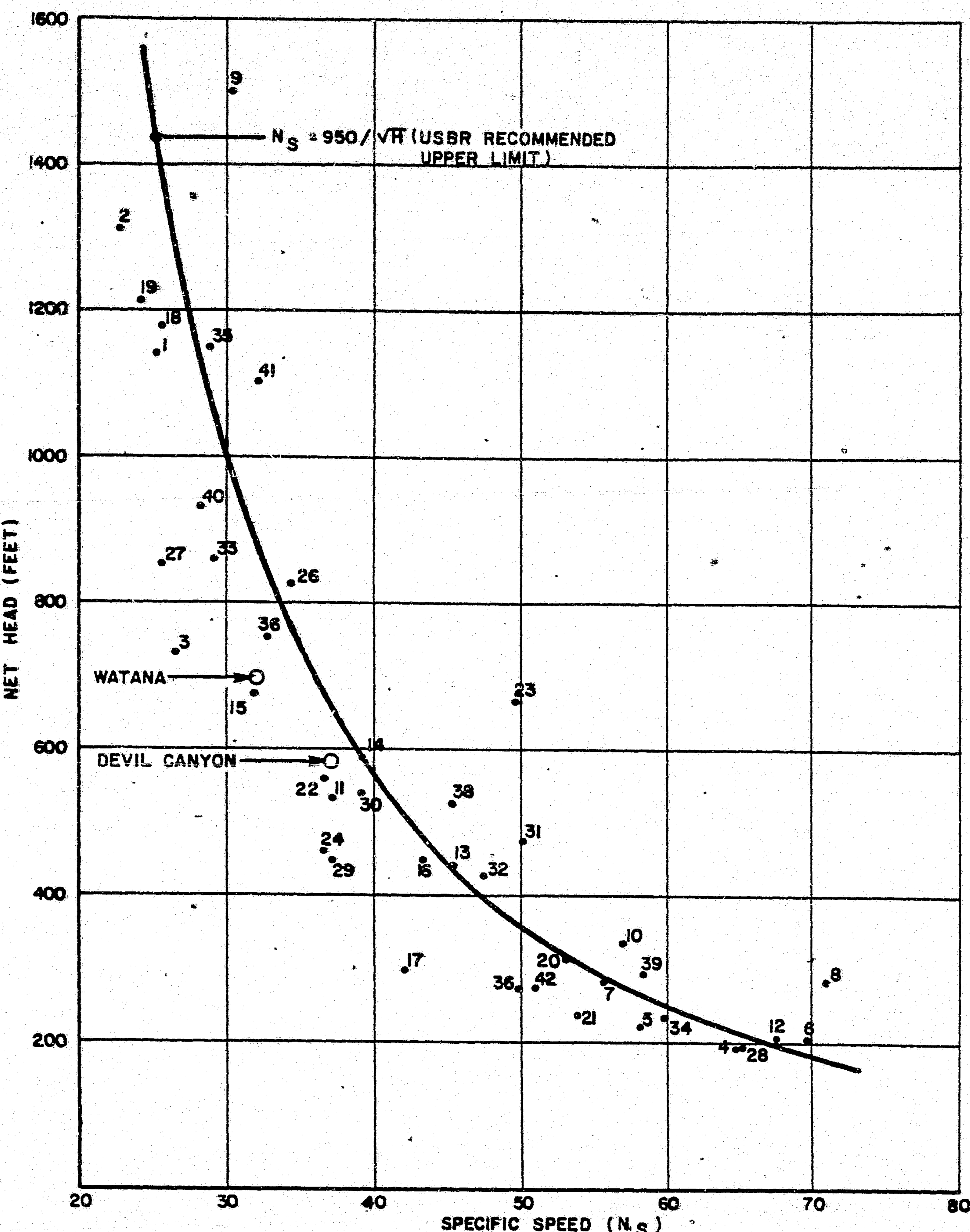
WATANA - UNIT OUTPUT

FIGURE 12.21



WATANA - TURBINE PERFORMANCE
(AT RATED HEAD)

FIGURE 12.22



FRANCIS TURBINES
SPECIFIC SPEED EXPERIENCE CURVE
FOR RECENT UNITS

FIGURE 12.23

13 - DEVIL CANYON DEVELOPMENT

This section describes the various components of the Devil Canyon development, including diversion facilities, emergency release facilities, main dam, primary outlet facilities, reservoir, main and emergency spillway, saddle dam, the power intake, penstocks, and the powerhouse complex, including turbines, generators, mechanical and electrical equipment, switchyard structures, and equipment and project lands.

A description of permanent and temporary access and support facilities is also included.

13.1 - General Arrangement

The evolution of the Devil Canyon general arrangement is described in Section 10. The Devil Canyon reservoir and surrounding area is shown on Plate 64. The site layout in relation to main access facilities and camp facilities is shown on Plate 66A. A more detailed arrangement of the various site structures is presented in Plate 64.

The Devil Canyon dam will form a reservoir approximately 31 miles long with a surface area of 7,800 acres and a total volume of 1,092,000 acre feet at a normal maximum operating elevation of 1455. The operating level of the Devil Canyon reservoir is controlled by the tailwater level of the upstream Watana development. During operation, the reservoir will be capable of being drawn down to a minimum elevation of 1405.

The dam will be a thin arch concrete structure with a crest elevation of 1465 and maximum height of 645 feet. The dam will be supported by mass concrete thrust blocks on each abutment. On the left bank, the generally lower ground surface level will require a substantial thrust. Adjacent to this thrust block, an earth- and rockfill saddle dam will provide closure to the left bank. The saddle dam will be a central core type generally similar in cross section to the Watana dam. The dam will have a maximum height above foundation level of approximately 260 feet.

During construction, the reservoir will be diverted around the main construction area by means of a single concrete-lined diversion tunnel 32 feet in diameter on the left bank of the river.

A power intake located on the right bank will comprise an approach channel in rock leading to a reinforced concrete gate structure. From the intake structure four penstocks, consisting of concrete-lined tunnels each 20 feet in diameter will lead to an underground powerhouse complex housing four Francis turbines each with a rated capacity of 150 MW and four semi-umbrella type generations each rated at 180 MVA. Access to the powerhouse complex will be by means of an unlined access tunnel approximately 3,200 feet long, as well as a vertical access shaft about 950 feet deep. Turbine discharge will be conducted to the river by means of a single 39-foot-diameter tailrace tunnel leading from a surge chamber downstream from the powerhouse cavern. Compensation flow pumps at the power plant will ensure suitable flow in the river between the dam and tailrace tunnel outlet portal. A separate transformer gallery just upstream from the powerhouse cavern will house six single-phase 15/345 KV transformers. The transformers will be connected by 345-KV, single-phase, oil-filled cable through a cable shaft to the switchyard at the surface.

The primary outlet facility will consist of seven individual outlet conduits located in the lower part of the main dam; it will be designed to discharge all floods with a frequency of 1:50 years or less. Each outlet conduit will have a fixed-cone valve similar to those provided at Watana to minimize undesirable nitrogen supersaturation in the flows downstream. Flows resulting from floods with a frequency greater than 1:50 years but less than 1:10,000 years will be discharged by a chute spillway on the right bank, also similar in design to that provided for Watana. An emergency spillway on the left bank will provide sufficient additional capacity to permit discharge of the PMF without overtopping the dam. An emergency-release, low-level outlet facility will allow lowering of the dam to permit emergency inspection or repair.

13.2 - Site Access

(a) Roads

At Devil Canyon the main access road will enter the site from the south. A low level bridge crossing the Susitna River will be located just upstream of the dam. In addition to the main access, several ancillary roads will be required to the camp, village, tank farm, borrow areas, and construction roads to the dam and all major structures. These roads, with the exception of temporary haul roads, are shown on Plate 66A.

The construction roads will be gravel-surfaced roads 40 feet wide with small radius curves and grades limited to 10 percent. Major cut and fill work will be avoided. A gravel pad approximately five feet thic will be required for the roads. This will provide a drivable surface and also will protect against the sporadic permafrost areas.

(b) Bridges

The existing low level bridge upstream of the dam will be used to cross the Susitna River during construction. This bridge will be used during abutment excavation. After construction of the cofferdams is complete, the crests of these structures will be used to cross the river.

After construction of the main dam is completed, the crest of the main dam will provide access across the Susitna River.

(c) Airstrip

A permanent airstrip will be located at the Watana site, approximately 30 miles west of the Devil Canyon site. This strip will be used for the Devil Canyon development. The airstrip will be capable of accommodating the C-130 Hercules aircraft, and will also accommodate small jet passenger aircraft.

(d) Access Tunnel

An access tunnel will be provided to the underground powerhouse and associated works. The main access tunnel will be concrete-lined and will be approximately 35 feet wide and 28 feet high. The tunnel will allow permanent access to the operating development and will also be utilized during construction as the main construction tunnel. The tunnel will have construction adits branching off to the various components of the development during construction.

(e) Access Shaft

A vertical 20-foot diameter access shaft with an elevator will also be provided for access to the underground facilities. The powerhouse access through this shaft will be at the opposite end to the access tunnel.

13.3 - Site Facilities

(a) General

The construction of the Devil Canyon development will require various facilities to support the construction activities throughout the entire construction period. Following construction, the planned operation and maintenance of the development will be centered at the Watana development; therefore, minimum facilities at the site will be required to maintain the power facility.

As described for Watana (Section 12), a camp and construction village will be constructed and maintained at the project site. The camp/village will provide housing and living facilities for 3,200 people during construction. Other site facilities include contractor's work areas, site power, services, and communications. Items such as power and communications and hospital services will be required for construction operations independent of camp operations.

It is planned to dismantle and demobilize the facility upon completion of the project. After demobilizing the site, the area will be reclaimed. It is planned to utilize dismantled buildings and other items from the Watana development as much as possible in the camp/village.

Since the Watana development will be in service during the construction period, electric power will be available. It is therefore planned to meet all heating requirements with electric heat and not with fuel oil, as is planned for the Watana development. The salvaged building modules from the Watana camp/village will be retrofitted for electric heat.

(b) Temporary Camp and Village

The proposed location of the camp/village is on the south bank of the Susitna River between the damsite and Portage Creek, approximately 2.5 (see Plate 66A) miles southwest of the Devil Canyon. The south side of the Susitna was chosen because the main access is from the south. South-facing slopes will be used for the camp/village location.

The camp will consist of portable woodframe dormitories for single status workers with modular mess halls, recreational buildings, bank, post office, fire station, warehouses, hospital, offices, etc. The camp will be a single status camp for approximately 2,900 workers.

The village, designed for approximately 320 families, will be grouped around a service core containing a school, gymnasium, stores, and recreation area.

The two areas will be separated by approximately 1/2 mile to provide a buffer zone between areas. The hospital will serve both the main camp and the village.

This camp location will separate living areas from the work areas by a mile or more and will keep travel time to work to less than 15 minutes for most personnel.

The camp/village will be constructed in stages to accommodate the peak work force as presented in Table 13.1. Table 13.1 also presents the camp/village facility design numbers. The facilities have been designed for the peak work force plus 10 percent for "turnover". The "turnover" includes provisions or buffers for overlap of workers, vacations, and visitors. The conceptual layouts for the camp/village are presented in Plates ___ and ___.

(i) Site Preparation

Both the camp and the village areas will be cleared in select areas for topsoil, and the topsoil will be stockpiled for future use in reclamation operations. At the village site, selected areas will be left with trees and natural vegetation intact.

Both the main camp and the village site have been selected to provide well-drained land with natural slopes of 2 to 3 percent. A granular pad varying in thickness up to 8 feet will be placed in selected areas at the main camp. This will provide a uniform working surface for erection of the high density housing and service buildings and will serve in certain areas to protect the permafrost where it underlies the camp. In the village area, a granular pad will be installed only as necessary to support the housing units and to provide a suitable base for construction of the temporary town-center buildings.

All roadways within the camp/village areas will be flanked by roadside ditches, with CMP culverts carrying water across the intersections. In general, drainage will be through construction of a surface network of ditches. Peripheral ditches will intercept overland flows from adjacent non-cleared land and carry it around the camps.

Runoff will ultimately be directed to existing drainage channels leading to the Susitna River for the village and the main camp.

(ii) Facilities

Construction camp buildings will consist largely of trailer-type factory-built modules assembled at site to provide the various facilities required. The modules will be fabricated with heating, lighting, and plumbing services, interior finishes, furnishings, and equipment. Trailer modules will be supported on timber cribbing or blocking approximately two feet above grade.

Larger structures such as the central utilities building, warehouses, and hospital will be pre-engineered, steel-framed structures with metal cladding.

The larger structures will be erected on concrete-slab foundations. The slab will be cast on a non-frost susceptible layer at least equal to the thickness of the annual freeze/thaw layer. Heated permawalks will connect the majority of the buildings and dorms.

The various buildings in the camp are identified on Plate 67.

(c) Site Power and Utilities

(i) Power

Electrical power will be required to maintain the camp/village and construction activities. A 345 kV transmission line and substation will be in service during the construction activities. Two transformers will be installed at the substation to reduce the line voltage to the distribution voltage. One of the transformers will be the same transformer used at the Watana development during its construction.

Power will be sold to the contractors by APA. The peak demand during the peak camp population year is estimated at 20 MW for the camp/village and 4 MW for construction requirements, thus totaling 24 MW of peak demand. The distribution system in the camp/village will be 34.5 kV.

(ii) Water

The water supply system will serve the entire camp/village and selected contractor's work areas. The water supply system will provide for potable water and fire protection. The estimated peak population to be served will be 3,950 (2,900 in the camp and 1,050 in the village).

The principal source of water will be the Susitna River, with a backup system of wells drawing on ground water. The water will be treated in accordance with the Environmental Protection Agency (EPA) primary and secondary requirements.

A system of pumps and constructed storage reservoirs will provide the necessary system demand capacity.

The water distribution system will be a ductile iron pipe system contained in utilidors as described in Section 12.3.

(iii) Waste Water

The waste water collection and treatment system will serve the camp/village. One treatment plant will serve the camp/village. Gravity flow lines with lift stations will be used to collect the waste water from all of the camp and village facilities. The "in-camp" and "in-village" collection systems will be run through the perma-walks and utilidors so that the collection system will always be protected from the elements.

At the village, an aerated collection basin will be installed to collect the sewage. The sewage will be pumped from this collection basin through a force main to the sewage treatment plant.

An aerated collection basin will be needed at the village to balance out the highly cyclic waste water flows.

Chemical toilets located around the site will be serviced by sewage trucks, which will discharge directly into the sewage treatment plant.

The sewage treatment system will be a biological system with lagoons. The system will be designed to meet Alaskan state water law secondary treatment standards. The lagoons and system will be modular to allow for growth and contraction of the camp/village.

The location of the treatment plant is shown on Plates ___ and ___. The location was selected to avoid unnecessary odors in the camp as described for Watana.

The sewage plant will discharge its treated effluent through a force main to the Susitna River. All treated sludge will be disposed of in a solid waste sanitary landfill.

(d) Contractor's Area

The contractors on the site will require office, shop and general work areas. Office space for the contractors has been provided and its location is shown on Plate 66A.

Partial space required by the contractors for fabrication shops, storage or warehouses, and work areas within the camp confines has been designated and is shown on Plate 66A. Additional space required for the aforementioned items will be located between the main camp and the main access road.

13.4 - Diversion

(a) General

Diversion of the river flow during construction will be accomplished with a single 32-foot diameter horseshoe shaped section diversion tunnel. The concrete-lined tunnel will be located on the left bank of the river, and will be 1,490 feet in length. The diversion tunnel plan and profile is shown on Plate 69.

The tunnel is designed to pass a flood with a return frequency of 1:25 years routed through the Watana Reservoir. The peak inflow will be 37,800 cfs. Routing effects are negligible and the peak flow that the tunnel will discharge will be 37,800 cfs. The maximum water surface elevation upstream of the cofferdam will be El 944. A rating curve is presented in Figure 13.1.

(b) Cofferdams

Due to the depth of alluvium present in the Susitna riverbed foundation, a grouted zone through the alluvium material to bedrock excavation and to sound rock in the abutment areas will be installed.

The depth of alluvium material in the riverbed ranges up to a maximum of 70 feet. The alluvium material consists of open-worked gravels with numerous cobbles and boulders. Due to the coarseness of the alluvium material, a grouted zone was selected rather than a slurry wall; it will be constructed through the closure dam and alluvium material to bedrock and will minimize the amount of seepage into the main dam excavation. The abutment areas will be cleared and grubbed with excavation of all material to sound rock prior to placement of any cofferdam material.

The upstream cofferdam will be a zoned embankment found on the closure dam (see Plate 69). The closure dam will be constructed to Elevation 915 based on a low water level of Elevation 910 and will consist of coarse material on the upstream side grading to finer material on the downstream side. When the closure dam is completed, the grouting will commence and the zone will be constructed to minimize seepage into the main dam foundation excavation.

The cofferdam from Elevation 915 to 947 will be a zoned embankment consisting of a central core, fine and coarse upstream and downstream filters, and rock and/or gravel shells with riprap on the upstream face.

The downstream cofferdam will be a closure dam constructed from Elevation 860 to 895 (see Plate 69). It will consist of coarse material on the downstream side grading to finer material on the upstream side. When the closure dam is completed, the grouted zone can be constructed in the finer material to minimize seepage into the main dam foundation excavation.

The upstream cofferdam crest elevation will have a 3 foot freeboard allowance for settlement and wave runup. Thermal hydraulic studies conducted showed the discharged waters from the Watana Reservoir will be 34°F when they pass through Devil Canyon. Thus, an ice cover will not form upstream of the cofferdam, and no freeboard allowance was made for ice.

(c) Tunnel Portals and Gates

A reinforced concrete gate structure will be located at the upstream end of the tunnel (see Plate 70). The portal and gate for the tunnel will be designed for an external pressure (static) head of 250 feet.

Two 30 feet high by 15 feet wide water passages will be located in the gate structure with guides for the diversion closure gates separated by a center concrete pier. Each gate will be a fixed wheel vertical lift gate operated by a wire rope hoist in an enclosed housing. The gate will be designed to operate with the reservoir at Elevation 950, an 80 foot operating head.

Stoplog guides will be installed in the diversion tunnel outlet portal, and stoplogs will be provided to permit dewatering of the diversion tunnel for plugging operations.

The stoplogs will have a downstream skinplate and upstream seals (relative to river flow) and will be arranged in suitable sections to facilitate relatively easy handling, with a mobile crane using a follower beam.

(d) Operation During Diversion

The tunnel will pass all flows from 800 cfs to the upper design flow of 37,800 cfs. The rating curve for the diversion tunnel is shown in Figure 13.1.

(e) Final Closure and Reservoir Filling

Upon completion of the concrete dam to an elevation sufficient enough to pass the environmental flows with the discharge values that are incorporated in the dam, the tunnel will be temporarily closed with the intake gates and construction of the permanent plug will commence. It is estimated it will take a year to completely place and cure the plug. During this time the upstream gate and intake structure will be designed for a reservoir elevation of 1130, which will create an external pressure of 250 feet.

The filling of the reservoir will take approximately 11 days to full reservoir operating elevation of 1455.

13.5 - Arch Dam

(a) General

The arch dam at Devil Canyon has been selected in preference to a rockfill dam for the reasons given in Appendix D5. The shape of the canyon is suited to an arch dam, with a crest length-to-height ratio of approximately 2.

The height of the dam will be approximately 650 feet, well within the range of heights of dams constructed elsewhere. A comparative list of some large arch dams constructed throughout the world is given in Table 13.2.

Because Alaska is a highly seismic area, the arch dam will be designed to withstand dynamic loadings from intense seismic shaking. Some dams constructed throughout the world in high earthquake areas include the 741-foot high El Cajon dam in Honduras, the 696-foot high Mohamed Reza Shah Pahlavi dam in Iran, and the 548-foot high Vidraru Arges dam in Rumania. The Vidraru Arges dam and the 372-foot high Pacoima dam in California have both withstood high earthquake loadings, with the latter experiencing a base ground acceleration of between 0.6 to 0.8 g.

Green Lake dam is presently being constructed to a height of 210 feet in Sitka, Alaska.

(b) Location

The arch dam will be located at the upstream end of the canyon at its narrowest point. The rock is outcropping or very close to the surface at the abutments, and the contours just downstream of the left abutment swing in toward the river so that the left side of the dam will be founded against the upstream side of a slight promontory.

The rock forming the right abutment rises several hundred feet above the dam crest but on the left side the rock surface rises only to Elevation 1400. It will be necessary to construct a mass concrete thrust block at this point to artifically form the bearing surface of the dam.

(c) Foundations

The arch dam will be founded on sound bedrock located 20 to 40 feet below the bedrock surface. The foundation will be excavated and trimmed beneath the dam so that no abrupt irregularities will occur at the foundations which could cause stress concentrations within the concrete. During excavation the rock will also be trimmed as far as is practical, to increase the symmetry of the centerline profile and provide a comparatively uniform stress distribution across the dam. Areas of dikes and the local areas of poorer quality rock will be excavated and supplemented with dental concrete.

The foundation will be consolidation grouted over its whole area, and a double grout curtain up to 300 feet deep will run the length of the dam and its adjacent structures as shown in Plate 75. Grouting will be done from a system of galleries which will be run through the dam and into the rock. Within the rock these galleries will also serve as collectors for drainage holes which will be drilled just downstream of the grout curtain and will collect any seepage passing the curtain.

High on the left abutment open cracks are evident in the rock; these will be excavated to sound rock and the excavated material replaced with concrete in the form of a deep thrust block. On the right abutment a mass concrete thrust block will be founded at the end of the dam to match the left block and improve the dam symmetry.

(d) Arch Dam Geometry

The philosophy and design of the dam is more thoroughly described in Appendix D, but is summarized herein. The dam geometry is shown in Plates 72 and 73. The crown section at the center of the river will be of a double curved cupola shape inclined downstream. The static load from the reservoir will be taken primarily in the arches; the three-dimensional stress action of the structure will tend to induce tension in the downstream face of the cantilever. This will be offset by the gravity forces of the overhanging section, which also will counteract any seismic loadings produced by downstream ground motion.

A two-center configuration will be adopted for the arches to counteract the slight assymetry of the valley and give a more uniform stress distribution across the dam. The arches will be formed by circles with centers located on the vertical axis plane running along the center of the canyon. The radii of the arches on the right and wider side of the canyon will be greater than those on the left, and the thrust will be directed more nearly normal to the rock abutment rather than parallel to the face, as would occur with a smaller radius arch. The radii of the intrados or downstream face will be smaller than those of the extrados, producing a thickening of the dam at the abutments where stresses would tend to be highest.

(e) Thrust Blocks

The thrust blocks are shown on Plate 74. The massive concrete block on the left abutment will be formed to take the thrust from the upper part of the dam above the existing sound rock level. The thrust block will also serve as a transition between the concrete dam and the adjacent rockfill saddle dam. The inclined end face of the block will abut and seal against the impervious saddle dam core, and it will be wrapped by the supporting rock shell.

A thrust block will also be formed high on the right abutment at the end of the dam and adjacent to the spillway control structure. The block will improve the symmetry of the dam profile, as previously stated, and will be stable under load conditions similar to those incurred by the thrust block on the left bank.

(f) Construction and Schedule

Construction of the dam will be completed over a five-year period as described in Section 17. Construction will take place throughout the year with cooling coils built into the concrete to dissipate the heat of hydration and special heating and insulation precautions taken in the winter to prevent excessive cooling of concrete surfaces. Concrete aggregates will be obtained from the alluvial deposits in the terraces upstream of the dam.

Concrete will be placed by means of three highlines strung above the dam between the abutments.

13.6 - Saddle Dam

The design philosophy for the saddle dam at Devil Canyon is essentially the same as that for the main dam at Watana described in Section 12.6. The most significant difference is the exclusive use of rockfill in the shells instead of river gravels used at Watana. The use of gravels in the upstream shell at Watana is to minimize settlement of the shell on saturation during filling of the reservoir and to ensure a free draining material. These aspects of the design are not as significant for the much smaller structure at Devil Canyon. The amount of settlement will be less and the drainage paths for the dissipation of any excess pore pressures will be much reduced. Many dams of equal or larger dimensions have been constructed of similar materials and the design is well within precedent.

(a) Proposed Dam Cross Section

Details of the proposed saddle dam are shown in Plate 76. As at Watana, the central vertical impervious core will be protected by fine and coarse filters on both upstream and downstream slopes and supported by rockfill shells. The core will have a crest width of 15 feet and side slopes of 1H:4V to provide a core thickness to dam height ratio slightly in excess of 0.5.

The wide filter zones will provide sufficient material for self-healing of any cracks which might occur in the core due to settlement or as the result of seismic displacement.

The saturated sections of both shells will be constructed of compacted clean rockfill, processed to remove fine material in order to minimize pore pressure generation and ensure rapid dissipation during and after a seismic event. Pore pressures cannot develop in the unsaturated section of the downstream shell and the material in that zone will be unprocessed rockfill from surface or underground excavations.

Protection on the upstream slope will consist of a 10-foot layer of rip-rap.

(b) Sources of Construction Material

No source of material suitable for the core of the saddle dam has been identified closer than the borrow areas at Watana (Areas D and H). The current proposals are to develop Area D for core material at Watana and, since access roads will be established to that area, the same source will be used for the Devil Canyon core. Investigations to date indicate that suitable material can be obtained from areas above the Watana reservoir level. In the unlikely event that insufficient material is available from Area D, then Area H would be developed. The in-place volume of core material is 306,000 cubic yards.

The filter material will be obtained from the river deposits (Area G) immediately upstream of the main arch dam at Devil Canyon. This area will also be exploited for concrete aggregates. The total volume available in Area G is estimated to be 6 million cubic yards, while the concrete aggregate demand is some 2.7 million cubic yards. The estimated volumes required for the dam are 228,000 and 181,000 cubic yards for the fine and coarse filters, respectively.

Rockfill for the shells will be obtained primarily from the excavations for the spillways, tunnels, and powerhouse complex. The total rockfill required will be approximately 1.2 million cubic yards. The proportion of sound rock suitable for use in the dam which can be obtained from the excavations cannot be accurately assessed at this stage, but it is proposed to make good any shortfall by deepening and extending the emergency spillway cut. This will be more economical and environmentally acceptable than developing quarry Area K, some 2 miles from the damsite.

(c) Excavation and Foundation Preparation

The excavation and foundation preparation will be as for the Watana site with all alluvium and other unconsolidated deposits under the dam removed to expose sound bedrock to eliminate any risk of liquefaction of the dam foundation under earthquake loading. Weathered and heavily jointed rock will be removed from beneath the core, and filters and local irregularities in the rock surface either trimmed back or concrete added to provide a suitable contact surface for placing the core.

(d) Grouting and Pressure Relief

As at Watana, the rock foundation will be improved by consolidation grouting over the core contact area and by a grouted cutoff along the centerline of the core. The cutoff at any location will extend to a depth of 0.7 the water head at that location as shown on Plate 75.

A grouting and drainage tunnel will be excavated in bedrock beneath the dam along the centerline of the core and will connect with a similar tunnel beneath the adjacent concrete arch dam. Pressure relief and drainage holes will be drilled from this tunnel and seepage from the drainage system will be discharged into the arch dam drainage system and ultimately downstream below tailwater level.

(e) Impervious Core and Filters

The requirements for impervious core and both fine and coarse filters will be as for the Watana dam (Section 12.6).

(f) Rockfill Shells

The processed rockfill to be placed in the saturated zones of the dam will have the same grading as the processed alluvium used at Watana. The maximum size shall not exceed 12 inches and not more than 10 percent of the material shall be finer than 3/8 inch size. This restriction on fine material will not apply to rockfill in the unsaturated zone above Elevation 1375 in the downstream shell. All rockfill will be placed and thoroughly compacted in 24-inch layers.

(g) Freeboard and Superelevation

The highest reservoir level will be Elevation 1466 under maximum probable flood (PMF) flows. At this elevation the fuse plug in the emergency spillway will be breached and the reservoir level will fall to the spillway sill elevation of 1434. The normal maximum pool elevation is 1455.

It is proposed that a minimum freeboard of three feet be provided for the PMF flood; hence, the crest of the saddle dam cannot be lower than Elevation 1469. In addition, an allowance of one percent of the height of the dam will be made for potential slumping of the rockfill shells under seismic loading. An allowance of one foot has been made for settlement of both abutments; hence, the final crest elevations of the saddle dam will be 1470 at the abutments, rising in proportion to the total height of the dam to Elevation 1472 at the maximum section. Under normal operating conditions, the freeboard will range from 15 feet at the abutments to 17 feet at the center of the dam. Further allowances must be made to compensate for static settlement of the dam after completion due to its own weight and the effect of saturation of the upstream shell, which will tend to produce additional breakdown of the rock fill at point contacts. It is proposed that one percent of the dam height be allowed for such settlement, giving a maximum crest elevation on completion of the construction of 1475 at the maximum height, and 1471 at the abutments.

The allowances for post-construction settlement and seismic slumping will be achieved by steepening both slopes of the dam above Elevation 1400.

(h) Instrumentation

Instrumentation will be installed within all parts of the dam to provide monitoring during construction as well as during operation. Instruments for measuring internal vertical and horizontal displacements, stresses and strains, and total and fluid pressures, as well as surface monuments and markers, will be installed. The quantity and location will be decided during final design. Instrumentation will include the following:

(i) Piezometers - To measure static pressure of fluid in the pore spaces of soil and rockfill.

(ii) Internal Vertical Movement Devices

- Cross-arm settlement devices as developed by the USBR;
- Various versions of the taut-wire devices developed to measure internal settlement; and
- Hydraulic settlement devices of various kinds.

(iii) Internal Horizontal Movement Devices

- Taut wire arrangements;
- Cross-arm devices;
- Inclinometers; and
- Strain meters.

(iv) Other Measuring Devices

- Stress meters;
- Surface monuments and alignment markers; and
- Seismographic recorders and seismoscopes.

(i) Stability Analyses

As at Watana, special precautions have been taken to ensure stability under earthquake loading by the use of processed free draining rockfill in the saturated zones of the dam, the incorporation of very wide filter zones, and the removal of all unconsolidated natural material from beneath the dam.

Static and dynamic stability analyses of the upstream slopes of the Watana dam (Section 12.6), have confirmed stable slopes under all conditions for a 24H:1V upstream slope and a 2H:1V downstream slope.

Since these same slopes have been used for the Watana dam and the construction materials are essentially similar, it was considered unnecessary to carry out further analysis for the specific details of the saddle dam to confirm feasibility, though such analyses will be required during the final design phase.

13.7 - Primary Outlet Facilities

(a) General

The prime function of the outlet facilities is to provide for discharge through the main dam of routed floods with up to 1:50 years recurrence period at the Devil Canyon reservoir. Downstream erosion is to be minimal and nitrogen supersaturation of the releases is to be restricted as much as possible as in the case of the Watana development. A further function of the releases is to provide an emergency drawdown for the reservoir, should maintenance be necessary on the main dam or low level submerged structures, and also to act as a diversion during the latter part of the construction period as described in Section 13.4.

The facilities will be located in the lower portion of the main dam, as shown on Plate 76A, and will consist of seven free discharge valves set in the lower part of the arch dam.

(b) Outlet

The discharge valves will be fixed cone valves located at two elevations: the upper grouping, consisting of four 102-inch diameter valves, will be set at Elevation 1050, and the lower grouping of three 90-inch diameter valves will be set at Elevation 930. The valves will be installed radially (normal to the dam centerline) with the upper set centered on a point slightly downstream of that corresponding to the lower valves.

The fixed cone valves will be installed on individual conduits passing through the dam, set from the downstream face, and protected by upstream ring follower gates located in separate chambers within the dam, as shown on Plate 76A. The gates will serve to isolate the valves to allow maintenance. Monorail hoists will be located above each valve and gate assembly to provide for their withdrawal and maintenance. The gates and valves will be linked by a 20 foot high tunnel running across the dam and into the left abutment where access will be provided by means of a vertical shaft exiting through the thrust block. Although secondary access will be provided via a similar shaft from the right abutment vehicle, access and installation are both considered to be from the left side.

The valve and gate assemblies will be protected by individual trashracks installed on the upstream face. The racks will be removable along guides running on the upstream dam face. The racks will be raised by a mobile crane normally stationed at Watana but employed for both sites.

(c) Fixed Cone Valves

The 102-inch diameter valves operating at a gross level of 420 feet and the 90-inch diameter valves operating at a head of 525 feet have been selected to be within current precedent considering the valve size and the static head on the valve. The valves will be located in individually heated rooms and will be provided with electric jacket heaters installed around the cylindrical sleeve of each valve. The valves will be capable of year round operation, although winter operation is not contemplated. Normally when the valves are closed, the upstream ring follower gates will also be closed to limit leakage and freezing of water through the valve seats.

The valves will be operated either by two hydraulic operators or by screw stem hoists. The former have been assumed for preliminary design purposes. The valves will normally be operated remotely from Watana, but local operation is also possible.

Thorough research and model studies will be required for final design of the valves, particularly in regard to preventing vibration. In sizing the valves it has been assumed that the valve gate opening will be restricted to 80 percent full stroke to reduce possibilities of vibration.

(d) Ring Follower Gates

Ring follower gates will be installed upstream of each valve and will be used:

- To permit inspection and maintenance of the fixed cone valves;

- To relieve hydrostatic pressure on the valves when they are in the closed position; and

- To close against flowing water in the event of malfunction or failure of the valves.

The ring follower gates will have nominal diameters of 102 and 90 inches and will be of welded or cast steel construction. The gates will be designed to withstand the total static head with full reservoir. Existing large diameter, high head ring follower gates are summarized in Table 12.6.

The ring follower gates will be designed to be lowered under flowing water conditions with maximum head, although normally they will be raised under balanced head conditions only. Valved bypass piping will be used to equalize the pressure on both sides of the ring follower gate before raising. The gates will be operated by hydraulic cylinders with a nominal operating pressure of 2,000 psi. Either local or remote operation of the ring follower gates will be possible.

A grease system will be installed in each gate for injection of grease between the gate leaf and the gate body seats to reduce frictional forces when the gates are operated.

(e) Trashracks

A steel trashrack will be installed at the upstream entrance to each water passage. The racks will prevent debris from being drawn into the discharge valves.

The bar spacing on the racks will be approximately 7 inches, and the racks will be designed for a maximum differential head of about 40 feet. The maximum net velocity through the racks will be approximately __ ft/s. Provision will be made for monitoring head loss across the racks.

(f) Bulkhead Gates

Provision will be made for installing intake bulkhead gates at the upstream entrance to each of the conduits for the fixed cone valves. Embedded guides will be installed at each conduit entrance extending to above maximum normal water level.

The bulkhead gates will be installed only under balanced head conditions using a gantry crane. The gates for the upper valves will be 12 feet square and will be 10 feet square for the lower valves.

Each gate will have a downstream skinplate and seal and will be designed to withstand full differential head with maximum reservoir water level. One gate of each size has been assumed and the gates will be stored at the dam crest level.

A temporary cover will normally be placed in the bulkhead gate check at trashrack level to prevent debris from getting behind the trashracks at the front of the valve conduit.

The crane for handling the bulkhead gates will be an electric travelling gantry type crane located on the main dam crest at Elevation 1468. The estimated crane capacity is 40 tons. The crane will have a single point lift hoist mounted on a moveable trolley. The hoist pickup will be incorporated into a follower for handling the bulkhead gates.

13.8 - Main Spillway

(a) General

The main spillway at Devil Canyon will be located on the right hand side of the canyon (see Plate 77). The upstream control structure will be adjacent to the arch dam thrust block and will discharge down an inclined concrete-lined chute, constructed down the steep face of the canyon, and then over the flip bucket which will traject flows downstream and into the river below.

The right side location for the spillway facilities is considered preferrable to the left because of the superior quality of the rock, with its lower degree of weathering, and the downstream alignment of the river, which allows for spillway discharges paralleling the direction of flow.

The spillway will be designed to pass the 1:10,000 year routed flood at Watana in conjunction with the outlet facilities, giving it a design capacity of 115,000 cfs, which will be discharged over a total head drop of 550 feet. No surcharge will occur above the normal maximum reservoir operating level of 1,455 feet. This will be below the discharge of 150,000 cfs over 600 feet on the Mica Project, developed for British Columbia Hydro, Canada, which has operated successfully for a number of years.

(b) Approach Channel and Control Structure

The approach channel will be excavated to a depth of approximately 100 feet in the rock with a width of just over 130 feet and an invert elevation of 1375, which will produce a flow velocity of approximately 11 ft/s under design discharge.

The control structure will be a three unit concrete structure set at the end of the channel. Each unit will house a 54 foot high by 30 foot wide gate which will sit on top of an ogee-crested weir and, in conjunction with the other gate units, will control the flows passing though the spillway. The gates will be fixed wheel gates operated by individual rope hoists.

Each gate will be contained with a separate monolith unit consisting of an ogee overflow weir, piers, and integral roadway deck. The box configuration of the unit will give the individual monoliths stability during earthquake motion, and the "split pier" construction, with each unit having its separate piers, will allow for some relative motion with no stress transference between units during seismic events and less chance of gate seizure. Model tests will be necessary during the final design stage to determine final geometry and dimensions of pier noses, crest slope, and pier lengths. The main access route will cross the dam and the control structure deck upstream of the gate hoists. The main dam grout curtain and drainage system will pass beneath the structure.

(c) Spillway Chute

The spillway chute will cut across the steep face of the canyon for a distance of approximately 900 feet and will terminate at Elevation 1000. The chute will taper uniformly over its length from 122 feet at the upstream end to 80 feet downstream.

The slope of the bedding planes on the right abutment above the chute will be at an approximate angle of 55° or more. Because of the instability along these planes, the rock above the spillway will be cut back parallel to the bedding and the face will be reinforced with steel mesh and rock bolts.

The chute itself will be concrete-lined with invert and wall slabs anchored back to the rock. The profile of the chute will be such that the invert slabs will be founded on sound rock. Part way down the chute on the side closest to the river the depth of cut will be insufficient to provide the supporting rock to the slabs; hence, the side wall will take the form of a gravity section over approximately a 200 foot length.

The velocity at the bottom of the chute will be approximately 150 ft/s. In order to prevent cavitation of the chute surfaces, air will be introduced into the discharges. Air will be drawn in along the chute via an underlying aeration gallery and offshoot ducts extending to the downstream side of a raised step running transverse to the chute, as shown on Plate 77.

The chute will be underlain by a series of box drains at the rock/concrete interface which will drain through drilled holes to an inclined rock tunnel below running the length of the spillway.

(d) Flip Bucket

The spillway chute will terminate in a mass concrete flip bucket founded on sound rock at Elevation 1000, approximately 100 feet above the river. The curve of the flow surface of the bucket will be adjusted to confine the issuing discharge, but at present it is assumed to be cylindrical and will be modified at the final design stage following model tests. A grouting drainage gallery will be provided within the bucket to allow for foundation consolidation and relief of uplift pressures.

The jet issuing from the bucket will be trajected downstream and parallel to the river below.

(e) Plunge Pool

The impact area of the issuing spillway discharge will be limited to the area of the downstream river surface to prevent excessive erosion of the canyon side walls. This will be done by modification of the flow surface of the flip bucket as described above. Over this impact area the alluvial material in the riverbed will be excavated down to sound rock to provide a plunge pool in which most of the inherent energy of the discharges will be dissipated, although some energy will already have been dissipated by friction in the chute, in dispersion, and friction through the air.

It was considered necessary to excavate the river material to determine the general area of any downstream erosion and also to prevent excessive erosion and random downstream deposition of material which might occur if discharges were allowed to excavate their own pool.

13.9 - Emergency Spillway

(a) General

The emergency spillway will be located on the left side of the river beyond the rockfill saddle dam. It will be set within the rock underlying the left side of the saddle and will continue downstream for approximately 2,000 feet.

An erodible fuse plug, consisting of impervious material and fine gravels, will be constructed at the upstream end and will be designed to wash out when overtopped by the reservoir, releasing floods of up to 160,000 cfs in excess of the combined main spillway and outlet capacities and thus preventing overtopping of the main dam.

(b) Fuse Plug and Approach Channel

The approach channel to the fuse plug will be cut in the rock and will have a width of 310 feet and an invert elevation of 2170. The channel will be crossed by the main access road to the dam on a bridge consisting of concrete piers, precast beams, and an in situ concrete bridge deck. The fuse plug will close the approach channel and will have a maximum height of 31.5 feet with a crest elevation of 1465.5. The plug will be located on top of a flat-crested weir excavated in the rock and protected with a concrete slab. Since the rock slopes quite steeply at the channel location, it is desirable to keep the spillway chute as narrow as possible to reduce the excavation quality. For this reason a drop section downstream of the plug will be introduced to increase the discharge coefficients at the plug sections and thus enable a reduction in the length of the plug.

The plug will be traversed by a pilot channel with an invert elevation of 1464, and will have a similar zoning to that described in Section 12.10 for Watana.

(c) Discharge Channel

The channel will narrow downstream and lead to a high tributary valley above the Susitna River. This channel will rapidly erode under high flows but will serve the purpose of straining the initial flows in the direction of the valley.

13.10 - Devil Canyon Power Facilities

(a) Intake

The intake structure is located on the right bank as shown on Plate 83. Separate intakes are provided for each of the units which will operate for reservoir levels between Elevations 1455 and 1405. Each intake has a single intake gate, a set of steel trashracks, and provision for placing a bulkhead upstream from the gate as shown on Plate 83. A traveling gantry crane on the intake deck at Elevation 1466 will service all four intakes. The mechanical equipment is described in more detail below.

The intake is located at the end of a 200-foot-long unlined approach channel. The structure is founded deep in the rock. The rock face between intakes will be lined with concrete to stabilize the rock surface. The grout curtain and drainage holes will continue beyond the main dam and beneath the structure.

The 60-foot spacing of the intakes was set similar to the spacing of the turbines, which will give parallel penstocks and allow for easier setting out of the tunnels during construction.

(b) Intake Gates

The four power intakes will have a single fixed wheel intake gate with a nominal operating size of 16 feet wide by 20 feet high. The gates will have an upstream skinplate and seal and will be operated by hydraulic or wire rope hoists located in heated enclosures immediately below deck level. The gates will normally close under balanced head conditions to permit dewatering of the penstock and turbine water passages for turbine inspection and maintenance. The gates will also be capable of closing under their own weight with full flow conditions and maximum reservoir water level in the event of loss of control of the turbines. A heated air vent will be provided at the intake deck to satisfy air demand requirements when the intake gate is closed with flowing water conditions.

(c) Intake Bulkhead Gates

One set of intake bulkhead consisting of two gate sections will be provided for closing the intake openings. The gate will be used to permit inspection and maintenance of the intake gate and intake gate guides. The gates will be raised and lowered under balanced water conditions only. To balance water pressure when raising the intake bulkhead, the space between the gate and the downstream control gate will be flooded by means of a follower-operated bypass valve on the top gate section. An air valve will be provided in the top of the gate. The gates will have a downstream skinplate and seal on the downstream side. They will be designated to withstand full differential pressure.

(d) Trashracks

Each of the four intakes will incorporate trashracks just upstream from the maintenance gates. The trashrack opening will have a bar spacing of about 6 inches and designed for a maximum differential head of about 30 feet. The maximum gross velocity through the racks will be about 4 ft/s. Each trashrack will be constructed in two sections for removal by means of a follower suspended from the intake gantry crane.

(e) Intake Gantry Crane

A 60-ton capacity (approximately) electrical traveling gantry crane will be provided on the intake deck at Elevation 1466 for handling the trashracks, maintenance gates and intake gates. The crane will incorporate a double point lift hoist mounted on an enclosed trolley. The hoist pickup will be incorporated into a follower for handling the gates and trashracks. The crane will also have a grappling hoist with a grapple of approximately 5-ton capacity for removing debris from in front of the trashrack.

13.11 - Penstocks

The power plant will have four penstocks, one for each unit. The maximum static head on each penstock is 638 feet, measured from normal maximum operating level (Elevation 1455) to centerline distributor level (Elevation 817). An allowance of 35 percent has been made for pressure rise in the penstock under transient conditions, giving a normal maximum design head of 861 feet. Maximum extreme head, corresponding to maximum reservoir flood level, is 876 feet.

The penstocks have been designed as individual concrete-lined rock tunnels from the intake to the powerhouse. The section 200 feet upstream of the powerhouse is steel lined. The inclined sections of the concrete-lined penstocks are at 55° to the horizontal.

(a) Steel Liner

It has been assumed that the rock adjacent to the powerhouse cavern will be incapable of long-term restraint against the forces transmitted from the penstock hydraulic pressures. The first 50 feet of steel liner will therefore be required to resist the design pressure without contributing from the surrounding concrete. For the remainder of the steel liner, which extendes a further 150 feet upstream, allowance is made for partial rock support to reduce the steel stress. For preliminary design, it is assumed that not more than 50 percent of the design head is taken by the rock support over this transition length.

Beyond the steel liner, the hydraulic loads are taken solely by the rock tunnel with a concrete liner.

The steel liner is surrounded by a concrete infill with a minimum thickness of 24 inches. A preliminary analyses has evaluated that the optimum internal diameter of the steel lining is 15 feet, based on the minimum total cost of construction and the capitalized value of energy reduction due to head loss. A tapering steel transition has been provided at the junction between the steel liner and the concrete liner to increase the internal diameter from 15 feet to 20 feet.

(b) Concrete Liner

The penstocks are fully lined with concrete from the intake to the steel lined section adjacent to the powerhouse; the thickness of lining will vary with the design head. The minimum thickness of lining is 12 inches.

Based on preliminary analyses, the optimum internal diameter of the concrete liner is 20 feet.

(c) Grouting and Pressure Relief

A comprehensive pressure relief system is required to protect the underground caverns against seepage from the high pressure penstocks. The system will comprise of small diameter boreholes set out in patterns and curtains to intercept the jointing in the rock.

Grouting round the penstocks will be provided to:

- fill and seal any voids between the concrete infill and the steel liner which may be left after the concrete placing and curing; and
- fill joints or fractures in the rock surrounding the penstocks to reduce flow into the pressure relief system and to consolidate the rock.

13.12 - Powerhouse

(a) General

The powerhouse complex will be constructed underground in the right abutment. This will require the excavation in rock of three major caverns (powerhouse, transformer gallery and surge chamber), with interconnecting rock tunnels for the draft tubes and isolated phase bus ducts.

An unlined rock tunnel will be required for vehicular access to the three main rock caverns. A second unlined rock tunnel will provide access from the powerhouse to the foot of the arch dam, for routine maintenance on the fixed cone valves; this tunnel will also provide construction access to the lower section of the penstocks.

Vertical shafts will be required for personnel access by elevator to the underground powerhouse; for oil filled cable from the transformer gallery, and for surge chamber venting.

The draft tube gate gallery and cavern are located in the surge chamber cavern, above maximum design surge level.

The general layout of the powerhouse complex is shown on Plates 85, 86 and 87. The transformer gallery is located upstream of the powerhouse cavern; the surge chamber is located downstream of the powerhouse cavern. The clear spacing between the underground caverns is at least 1.5 times the main span of the larger excavation, from geotechnical considerations.

(b) Layout Considerations

The powerhouse is located underground in the right abutment. Water for power generation is taken from an intake structure to the right of the main spillway, and carried through individual penstocks to the turbines. Water is discharged to the river by a single tailrace tunnel 6800 feet in length. The draft tubes and tailrace tunnel are protected against excessive transient pressure rise by a downstream surge chamber, which also provides storage for the turbine start-up sequence.

The intake structure is designed for a maximum drawdown of 50 feet and is located close to the main arch dam thrust block for ease of access. The powerhouse is located to provide the minimum total length of penstock, assuming an inclination of 55° to the horizontal for the sloping section of penstock. The orientation of the powerhouse has been selected as a compromise between the desired orientation for power flow (E-W) and the geotechnical data on known shear zones and joint sets. Minimum clear spacing between major rock excavations is at least 1.5 times the span of the larger excavation. This is considered a conservative estimate for preliminary design purposes.

The downstream surge chamber will be constructed as close as possible to the powerhouse for maximum protection to the draft tubes under transient load conditions. For this reason the underground transformer gallery will be located upstream from the powerhouse. The rock around the powerhouse cavern and transformer gallery is protected against high pressure seepage from the penstocks by a 200-foot continuous steel-lining and an extensive pressure relief system.

(c) Access Tunnels and Shafts

Vehicle access to the underground facilities at Devil Canyon is provided by two unlined rock tunnels. The main access tunnel, 3,000 feet long, connects the powerhouse cavern at Elevation 852 with the canyon access road on the right bank. A secondary access tunnel runs from the main powerhouse access tunnel to the foot of the arch dam, for routine maintenance of the fixed cone valves. A branch tunnel from the secondary access tunnel will provide construction access to the lower section of the penstocks, at Elevation 820. Separate branch tunnels from the main access tunnel give vehicle access to the transformer gallery at Elevation 896 and the draft tube gate gallery at Elevation 908. The maximum gradient on the permanent access tunnel is 8 percent; the maximum gradient on the secondary access tunnel is 9 percent.

The cross section of the access tunnels is dictated by requirements for construction plant; for preliminary design a modified horseshoe shape 35-feet wide by 28-feet high has been used.

The main access shaft is located at the north end of the powerhouse cavern, providing personnel access by elevator from the surface. Horizontal tunnels are provided from this shaft for pedestrian access to the transformer gallery and the draft tube gate gallery. At a higher level access is also available to the fire protection head tank.

Access to the upstream grouting gallery is from the transformer gallery main access tunnel, at a maximum gradient of 13.5 percent.

(d) Powerhouse Cavern

The main powerhouse cavern is designed to accommodate four vertical shaft Francis turbines, in line, with direct coupling to overhung generators. Each unit is designed to generate 164 MW at 575-foot head.

The overall height of the cavern is governed by the physical size of the turbine and generator, space requirements for miscellaneous equipment and services, the design dimensions of the turbine draft tube, the overhead travelling crane clearance and size, and the rise of the roof arch. The selected unit spacing is 60 feet; in addition, a 110-foot service bas has been allowed at the south end of the powerhouse for routine maintenance and construction erection. The local control room is located at the north end of the main powerhouse floor. The width of the cavern allows for the physical size of the generator plus galleries for piping, air-conditioning ducts, electrical cables, and isolated phase bus. The penstock steel-liner is continuous with the turbine spiral case; no penstock inlet valves are provided in the powerhouse. Continuous drainage galleries are provided to a low-level sump.

Compensation flow of 500 ft^3/s is required to the river immediately downstream of the arch dam, in view of the length of the tailrace tunnel (6800 feet). This flow is provided by two No. 1300 hp vertical shaft mixed flow pumps, installed in a gallery below the service bay. Each pump is rated at 115,000 gpm at 35-foot head. Water is taken from the base of the surge chamber and pumped 1000 feet to the dam through a steel pipe laid partly in the secondary access tunnel and partly in a separate outlet tunnel.

Multiple stairway access points are available from the powerhouse main floor to each gallery level. Access to the transformer gallery from the powerhouse is by a tunnel from the main access shaft or by a stairway through each of the four isolated phase bus shafts. Access is also available to the draft tube gate gallery by a tunnel from the main access shaft.

A service elevator is provided for access from the service bay area on the main floor to the machine shop, and the pumping and drainage galleries on the lower floors. Hatches have been provided through all main floors for installation and routine maintenance of pumps, valves and other heavy equipment using the main overhead travelling crane.

(e) Transformer Gallery

The transformers are located underground in a separate unlined rock cavern, 120 feet upstream of the powerhouse cavern, with four interconnecting tunnels for the isolated phase bus. There are 12 single-phase transformers in four groups of 3, one group for each generating unit. Each transformer is rated at 13/345, 70 MVA. For increased reliability, one spare transformer and one spare HV circuit are provided. The station service transformers (2x2 MVA) and the surface facilities transformers (2 x 7.5/10 MVA) are located in the bus tunnels. Generator excitation transformers are located on the main powerhouse floor.

High voltage cables are taken to the surface in two cable shafts, each 7 feet-6 inch internal diameter; provision is made for an inspection hoist in each shaft.

Vehicle access to the transformer gallery is from the south end via the main powerhouse access tunnel. Personnel access is from the main access shaft or through each of the four isolated phase bus tunnels.

(f) Surge Chamber

A simple surge chamber has been provided 120 feet downstream of the powerhouse to control pressure rise in the turbine draft tubes and tailrace tunnel under transient load conditions, and to provide storage for the machine start-up sequence. The chamber is common to all four draft tubes and the inlet pipe to the compensation flow pumps. The chamber design is governed by an assumed full load rejection surge and the requirement for incipient stability under part load operation, together with estimated floor levels from the tailwater rating curve.

The draft tube gate gallery and crane are located in the same cavern, above the maximum anticipated surge level. Access to the draft tube gate gallery is by a rock tunnel from the main access tunnel. The tunnel is widened locally for storage of the draft tube gates.

The chamger is generally an unlined rock excavation with localized rock support as necessary for stability of the roof arch and walls. The guide blocks for the draft tube gates will be of reinforced concrete anchored to the rock excavation by rockbolts.

13.13 - Reservoir

The Devil Canyon reservoir, at a normal operating level of 1455 feet, will be approximately 26 miles long with a maximum width in the order of 1/2 mile. The total surface area at normal operating level is 7800 acres. Just upstream of the dam, the maximum water depth will be approximately ___ feet. The minimum reservoir level will be 1405 feet during normal operation, resulting in a maximum drawdown of 50 feet. The reservoir will have a total capacity of 1,090,000 acre feet of which 420,000 acre feet will be live storage.

Prior to reservoir filling, the area below elevation 1460, five feet above maximum operating level, will be cleared of all trees and brush. A field reconnaissance of the proposed reservoir area was undertaken as part of these studies. This work included examination of aerial photographs and maps, an aerial overflight of the reservoir and collection of recent (1980 field season) forest inventory data from the U.S. Forest Service. As described for the Watana reservoir, most of the vegetatal material within the reservoir consists of trees with very little in brush. The trees are quite small, and the stands are not very dense. In the Watana reservoir area, an estimated 3,200,000 cubic feet of wood exists averaging approximately 500 cubic feet of low commercial quality, and some very significant logging problems would be posed by the steep slopes and incised terrain excavated. Approximately 87 percent of the available timber are soft woods. The results of the timber reconnaissance studies are described in more detail in Appendix C3.

The combination of steep terrain, moderate-light tree stocking levels, small trees, erosive potential of the reservoir slopes, remoteness, and very restricted access to the reservoirs are major factors affecting the choice of harvesting systems to be utilized for this project as discussed in Section 13.14.

Present market demand for the timber at Susitna is low, however, the worldwide demand for wood fluctuates considerably. It is anticipated that use of the harvested material would be limited to either sale as wood-waste products and as fuel.

Slash material including brush and small trees, which will be suitable for either of the above uses, will be either burned in a carefully controlled manner consistent wth applicable laws and regulations, or hauled to a disposal site in and adjacent to the reservoir. Material placed in disposal areas will be covered with an earthfill cover sufficient to prevent erosion and subsequent exposure.

A number of unstable areas will undoubtedly result during reservoir operation. These areas will require remedial treatment depending on the nature and extent of the instability.

13.14 - Tailrace Tunnel

The tailrace pressure tunnel is provided at Devil Canyon to carry power plant discharge from the surge chamber to the river. The tunnel has a modified horseshoe cross-section with a major internal dimension of 38 feet, and for preliminary design, it is assumed to be concrete lined throughout with a minimum thickness of 12 inches. The length of the tunnel is 6800 feet.

The size of the tunnel was selected after an economic study of the cost of construction and the capitalized value of average annual energy losses caused by friction, bends and changes of section. Since the size of the surge chamber is related to the effective diameter of the tailrace tunnel, the cost of the surge chamber was also included in the optimization studies as a function of tunnel size.

The tailrace portal site has been located at a prominent steep rock face on the right bank of the river to provide the required tunnel cover (about 60 feet) in as short a distance as possible. The portal provides a gradual transition from the tunnel modified horseshoe shape to a rectangular cross-section at the outlet; it also reduces the maximum outlet velocity to 8 ft/sec, to reduce the velocity head loss at exit. Vertical stoplog guides are provided for closure of the tunnel if required for tunnel inspection and/or maintenance.

13.15 - Turbines and Generators

(a) Unit Capacity

The Devil Canyon powerhouse will have four generating units with a nominal capacity of 150 MW. This is the available capacity with minimum December reservoir level (El. 1393) and a corresponding gross head of 553 feet in the station.

The head on the plant will vary from 605 feet maximum (597 feet net head) to 550 feet minimum (538 feet net head). Because maximum turbine output varies approximately with the 3/2 power of head, the maximum unit output will change with head as shown in Figure 13.2.

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

The generator rating has been selected as 180 MVA with a 90 percent power factor, which corresponds to a power output of 162 MW. 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 units will be operated at or near full load a large percentage of the time. The generators have therefore been 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.

(b) Turbines

The turbines will be of the vertical shaft Francis type with steel spiral casing and a concrete elbow type draft tube. The draft tube will have a single water passage (no center pier).

The rated output of the turbines will be 225,000 hp at 575 feet rated net head. Maximum and minimum heads on the units will be 597 feet and 538 feet, respectively. The full gate output of the turbines will be about 240,000 hp at 597 feet net head and 205,000 hp at 584 feet 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. This will be reviewed at the time of preparation of bid documents for the turbines.

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 13.3.

A speed of 225 rpm has been selected for the unit for preliminary design purposes. The resulting turbine specific speed (N_s) is 37.9. As shown in Figure 12.23, this is within present day practice for turbines operating under 575 feet head. The considerations for selection of turbine speed are briefly discussed in Section 12.16.

On the basis of information from turbine manufacturers and the studies on the power plant layout, the centerline of the turbine distributor has been set at 30 feet below minimum tailwater level. The final setting of the unit will be established in conjunction with the turbine manufacturer after the contract for the supply of the turbine equipment has been awarded.

The mechanical/structural designs of the turbines will be basically the same as for Watana. Because of the relatively short penstocks and the surge tank location immediately downstream from the powerhouse, the hydraulic transient characteristics of the turbines are favorable. Assuming normal generator inertia (H = 3.5 MW-Sec/MVA), a preliminary analysis has indicated the following:

- Water starting time (Tw) .... 1.2 sec.
- Mechanical starting time (Tm) .... 7.6 sec.
- Regulating ratio (Tm/Tw) 6.3
- Governor time .... 5.0 sec.
- Speed rise on full load rejection .... 35 percent
- Penstock pressure use on full load rejection .... 20 percent

The regulating ratio is above the minimum recommended by the USBR for good regulating. Also, unit speed rise and penstock capacity pressure rise are within normal accepted values. Because of the relatively short distance between the turbine and the tailrace surge tank and the deep unit setting, there should not be any problems with draft tube column separation.

As discussed in Section 12.16 for Watana, the units will be capable of operation from about 50 to 100 percent load. Considerations for draft tube surges and corresponding power swings as mentioned for Watana also will apply to Devil Canyon.

As with Watana, the relationship between generator natural frequency and the possible draft tube surge frequency is desirable and will require study in later design stages. Because of the high capacity factor for the Devil Canyon units, part load operation for these turbines is not as critical as at Watana; therefore, the possibility of problems with power swings is somewhat less of a concern than at Watana.

(c) Generators

The four generators in the Devil Canyon powerhouse will be of the vertical shaft, overhung type directly connected to the vertical Francis turbines.

The generators will be similar in construction and design to the Watana generators and the general features described in Section 12.16 for the stator, rotor, excitation system, and other details which apply for the Devil Canyon generators.

The rating and characteristics of the generators are as follows:

Rated Capacity:	180 MVA, 0.9 power factor with overload rating of 115 percent.
Rated Power:	162 MW
Rated Voltage:	15 kV, 3 phase, 60 Hertz
Synchronous Speed:	225 rpm
Inertia Constant:	3.5 kW - Sec/kVA
Short Circuit Ratio:	1.1 (minimum)
Efficiency at Full Load:	98 percent (minimum)

(d) Governor System

A governor system with electric hydraulic governor actuators will be provided for each of the Devil Canyon units. The system will be the same as for Watana.

13.16 - Miscellaneous Mechanical Equipment

(a) Compensation Flow Pumps

The two pumps for providing minimum discharge into the Susitna River between the dam and the tailrace tunnel outlet portal will be vertical mixed flow or axial type located in the powerhouse service bay below the main erection floor, as shown on Plate 87. Each pump will be rated at 250 cfs (115,000 gal/min) at 35 feet total head, and will be driven by 1,400-hp induction motors. The preliminary pump and motor data is summarized in Table 13.3.

A single pump intake will be located in the surge chamber with an 8-foot-diameter intake tunnel leading to the powerhouse. The intake tunnel will bifurcate into individual pump intake conduits within the powerhouse. The pump discharges will converge into a single pump discharge tunnel.

Butterfly type valves will be installed in the intake and discharge lines of each pump to permit isolation of a pump for inspection or maintenance. Trash screen guides and a trash screen will be provided in the surge chamber at the pump intake. It will be possible to remove the trash screen using the draft tube gate crane discussed below. The width of the guides will be selected so that one of the turbine draft tube gates may be installed in the intake to permit dewatering the pump intake tunnel for inspection and/or maintenance of the tunnel or the intake butterfly valves. Stoplog guides and a set of stoplogs will also be provided at the downstream end of the pump discharge tunnel to allow the discharge tunnel to be dewatered. The stoplogs will be handled with a mobile crane and a follower.

Pumping operation will be continuous; therefore, pumping equipment will be conservatively designed to provide efficient operation with minimal maintenance. Crane access will be provided for the pumps, motors, and valves to permit equipment servicing.

In the detailed design stages, consideration should also be given to turbine-driven rather than electric motor-driven pumps. A header from at least two of the main turbine penstocks would supply water to the turbines, with the turbine draft tubes connected to the pump discharge.

(b) Powerhouse Cranes

Two overhead type powerhouse cranes will be provided at Devil Canyon as at Watana. The estimated crane capacity will be 200 tons.

(c) Draft Tube Gates

Draft tube gates will be provided to permit dewatering of the turbine water passages for inspection and maintenance of the turbines. The arrangement of the draft tube gates will be the same as for Watana, except that only two gates will be provided, each 21 feet by 21 feet. At the time of starting of Unit 1, one gate will be installed in Unit 4 with the other gate available for Unit 1. Bulkhead domes will be installed in Units 2 and 3.

(d) Draft Tube Gate Crane

A crane will be installed in the surge chamber for installation and removal of the draft tube gates. The crane will be either a monorail (or twin monorail) crane or a gantry crane. For the preliminary design, a twin monorail crane of approximately 25-ton capacity has been assumed. The crane will be pendant-operated and have a two point lift. A follower will be used with the crane for handling the gates. The crane runway will be located along the upstream side of the surge chamber and will extend over the intake for the compensation flow pumps, as well as a gate unloading area at one end of the surge chamber.

(e) Miscellaneous Cranes and Hoists

In addition to the powerhouse cranes and draft tube gate cranes, the following cranes and hoists will be provided in the power plant:

- A 5-ton monorail hoist in the transformer gallery for transformer maintenance;

- Small overhead jib, or A-frame type hoists in the machine shop for handling material; and

- A-frame or monorail hoists in other powerhouse areas for handling small equipment.

(f) Elevators

Access and service elevators will be provided for the power plant as follows:

- Access elevator from the control building to the powerhouse;
- Service elevator in the powerhouse service bay; and
- Inspection hoists in cable shafts.

The elevators will be as discussed in Section 12.17 for Watana.

(g) Power Plant Mechanical Service Systems

The power plant mechanical service systems for Devil Canyon will be essentially the same as discussed in Section 12.17 for Watana, except for the following:

- There will be no main generator breakers in the power plant; therefore, circuit breaker air will not be required. The high-pressure air system will be used only for governor as well as instrument air. The operating pressure will be 600 to 1,000 psig depending on the governor system operating pressure. An air-conditioning system will be installed in the powerhouse control room.

- For preliminary design purposes only, one drainage and one dewatering sump have been provided in the powerhouse. The dewatering system will also be used to dewater the intake and discharge lines for the compensation flow pumps.

(h) Surface Facilities Mechanical Service Systems

The entrance building at the top of the power plant will have only a heating and ventilation system. The mechanical services in the standby power building will include a heating and ventilation system, a fuel oil system, and a fire protection system, as at Watana.

(i) Machine Shop Facilities

A machine shop and tool room will be located in the powerhouse service bay area to take care of maintenance work at the plant. The facilities will not be as extensive as at Watana. Some of the larger components will be transported to Watana for necessary machinery work.

13.17 - Accessory Electrical Equipment

(a) General

The accessory electrical equipment described in this section includes the following main electrical equipment:

- Main generator step-up 15/345 kV transformers;
- Isolated phase bus connecting the generator and transformers;
- 345 kV oil-filled cables from the transformer terminals to the switchyard;
- Control systems; and
- Station service auxiliary ac and dc systems.

Other equipment and systems described include grounding, lighting system and communications.

The main equipment and connections in the power plant are shown in the Single Line Diagram, (Plate 88). The arrangement of equipment in the powerhouse, transformer gallery, and cable shafts is shown in Plates 85 to 88.

(b) General Design Considerations for Transformers and HV Connections

(i) General

Twelve single-phase transformers and one spare transformer will be located in the transformer gallery. Each bank of the three single-phase transformers will be connected to one generator by isolated phase bus located in bus tunnels. The HV terminals of the transformer will be connected to the 345 kV switchyard by 345 kV single-phase oil-filled cables installed in 800-foot long vertical shafts. There will be two sets of three single-phase 345 kV oil-filled cables installed in each cable shaft. One additional set will be maintained as a spare three-phase cable circuit in the second cable shaft. These cable shafts will also contain the control and power cables between the powerhouse and the surface control room, as well as emergency power cables from the diesel generators at the surface to the underground facilities.

As described in Section 12.18 for the Watana power plant, a number of considerations led to the choice of the above optimum system of transformation and connections. Different alternative methods and equipment designs were also considered. In summary, these are:

- One transformer per generator versus one transformer for two generators;
- Underground transformers versus surface transformers;
- Direct transformation from generator voltage to 345 kV versus intermediate step transformation to 230 kV or 161 kV, and thence to 345 kV;
- Single phase versus three-phase transformers for each alternative method considered; and
- Oil-filled cable versus solid dielectric cable or SF_6 gas-insulated bus.

Reliability considerations are based on the general reliability requirements for generation and transmission described in Section 15 regarding the forced outage of a single generator, transformer, bus or cable in addition to planned or scheduled outages in a single contingency situation, or a subsequent outage of equipment in the double contingency situation. In the first case, the system should be capable of readjustment after the outage for loading within normal ratings and, in the second case, within emergency ratings.

The one transformer per generator scheme was selected since the operation of the Devil Canyon power plant will essentially be a continuous base-load type operation; also the smaller number of units at Devil Canyon compared to Watana will allow a transformer gallery of reasonable length for a unit generator-transformer scheme.

As at Watana, transport limitations for both dimensions and weight will preclude the use of the larger size three-phase transformers; hence, single-phase transformers will be used. One distinct advantage of single-phase transformers is that a spare transformer can be provided at a fairly low incremental cost.

For the same reasons as given in Section 12.18 for Watana, surface transformers and the double-step transformation scheme (15/161 kV generator-transformer, 161 kV cable and 161/345 kV auto-transformer at the switchyard) were ruled out. The direct transformation (15/345 kV) scheme with 345 kV oil-filled cables is considered a better overall scheme.

(c) Main Transformers

The transformers will be of the single phase, two-winding, oil-immersed, forced-oil water-cooled (FOW) type. A total of twelve single-phase transformers and one spare transformer will be provided, with rating and characteristics as follows:

Rated capacity:	70 MVA
High Voltage Winding:	345/ 3 kV, grounded Y
Basic Insulation Level (BIL) of HV Winding:	1300 kV
Low Voltage Winding:	15 kV, Delta
Transformer Impedance:	15 percent

The design and construction details are identical to the transformers at Watana as described in Section 12.18.

(d) Generator Isolated Phase Bus

Isolated phase bus connections will be located between the generator and the main transformer. The bus will be of the self-cooled, welded aluminum tubular type with design and construction details generally similar to the bus at the Watana power plant. The rating of the main bus is as follows:

Rated current:	9,000 amps
Short circuit current momentary:	240,000 amps
Short circuit current symmetrical:	150,000 amps
Basic Insulation Level (BIL):	150 kV

(e) <u>345 kV Oil-Filled Cable</u>

The general design considerations leading to the choice of the 345 kV oil-filled cable for the connections between the transformer HV terminals and the 345 kV switchyard at the surface are the same as described in Section 12.18 for the Watana plant.

The cables will be rated for a continuous maximum current of 400 amps at 345 kV ± 5 percent. The cables will be of single-core construction with oil flowing through a central oil duct within the copper conductor. The cables will be installed in the 800-foot cable shafts from the transformer gallery to the surface. No cable jointing will be necessary for this installation length.

(f) <u>Control Systems</u>

(i) <u>General</u>

The Devil Canyon power plant will be designed to be operated as an unattended plant. The plant will be normally controlled through supervisory control from the Susitna Area Control Center at Watana. The plant will, however, be provided with a control room with sufficient control, indication, and annunciation equipment to enable the plant to be operated during emergencies by one operator in the control room. In addition, for the purpose of testing and commissioning and maintenance of the plant, local control boards will be mounted on the powerhouse floor near each unit.

Automatic load-frequency control of the four units at Devil Canyon will be accomplished through the central computer-aided control system located at the Watana Area Control Center.

The power plant will be provided with "black start" capability similar to that provided at Watana, to enable the start of one unit without any power in the powerhouse or at the switchyard, except that provided by one emergency diesel generator. After the start-up of one unit, auxiliary station service power will be established in the power plant and the switchyard; the remaining generators can then be started one after the other to bring the plant into full output within the hour.

As at the Watana power plant, the control system will be designed to permit local-manual or local-automatic starting, voltage adjusting, synchronizing, and loading of the unit from the powerhouse control room at Devil Canyon.

The protective relaying system is shown in the main single line diagram, Plate 88, and is generally similar to that provided for the Watana power plant.

(g) Station Service Auxiliary AC and DC Systems

(i) AC Auxiliary System

The station service system will be designed to achieve a reliable and economic distribution system for the power plant and the switchyard and surface facilities. The auxiliary system will be similar to that in the Watana power plant except that the switchyard and surface facilities power will be obtained from a 4.16 kV system supplied by two 5/7.5 MVA, OA/FA, oil-immersed transformers connected to generators Nos. 1 and 4, respectively. The 4.16 kV double-ended switchgear will be located in the powerhouse. It will have a normally-open tie breaker which will prevent parallel operation of the two sections. The tie breaker will close on failure of one or the other of the incoming supplies. The 1400 hp compensation flow pumps will be supplied with power directly from the 4.16 kV system. Two 4.16 cables installed in the cable shafts will supply power to the surface facilities.

The 480 V station service system will be exactly similar to the Watana system described in Section 12.18, and will consist of a main 480 V switchgear, separate auxiliary boards for each unit, an essential auxiliaries board, and a general auxiliaries board. The main 480 V switchgear will be supplied by two 2000 kVA, 15,000/480 V grounded wye sealed gas dry-type transformers. A third 2000 kVA transformer will be maintained as a spare.

Two emergency diesel generators, each rated 500 kW, will be connected to the 480 V powerhouse main switchgear and 4.16 kV surface switchboard, respectively. Both diesel generators will be located at the surface.

An uninterruptible high-security power supply will be provided for the supervisory computer-aided plant control systems.

(ii) DC Auxiliary Station Service System

The dc auxiliary system will be similar to that provided at the Watana plant and will consist of two 125 V dc lead-acid batteries. Each battery system will be supplied by a double rectifier charging system. A 48 V dc battery system will be provided for supplying the supervisory and communications systems.

(iii) Black Start Capability

As at the Watana power plant, the Devil Canyon power plant will be provided with "black start" capability which will enable the plant to start up in a completely "blacked out" condition of the power plant and/or the power system.

(h) Other Accessory Electrical Systems

The other accessory electrical systems including the grounding system, lighting system, and powerhouse communications system will be similar in general design and construction aspects to the systems described in Section 12.18 for the Watana power plant.

13.18 - Switchyard Structures and Equipment

To follow

13.19 - Project Lands

Project lands acquired for the project will be the minimum necessary to construct access and site facilities, construct permanent facilities, to clear the reservoir, and to operate the project.

Appendix C contains land status background information relative to the Susitna Project, together with an inventory of private and public lands required for the project. A large amount of public land in the Devil Canyon area is managed by the Bureau of Land Management. There are large blocks of private Native Village Corporation Lands along the river. Other private holdings consist of widely scattered remote parcels. The state has selected much of the federal land in this area and is expected to receive a patent.

TABLE 13.1: WATANA PEAK WORK FORCE AND CAMP/VILLAGE DESIGN POPULATION

Calendar Year	Yearly Peak Force	Camp/Village Design
1992	180	200
1993	730	800
1994	1635	1800
1995	2455	2700
1996	3180	3500
1997	3180	3500
1998	2000	2200
1999	770	850
2000	455	500

TABLE 13.2: ARCH DAM EXPERIENCE

Dam	Location	Height ft(m)	Crest Length ft(m)
Inguiri (1985)	Georgia, USSR	892 (272)	2,513 (766)
Vaiont (1961)	Veneto, Italy	858 (262)	624 (190)
Mauvoisin (1957)	Valais, Switzerland	777 (237)	1,706 (520)
Chirkei (1975)	North Caucusas, USSR	764 (233)	1,109 (338)
El Cajon (1964)	Yoro/Cortes, Honduras	741 (226)	1,253 (382)
Contra (1965)	Ticino, Switzerland	722 (220)	1,246 (380)
Glen Canyon (1964)	Arizona, USA	710 (216)	1,560 (475)
Mohamed Reza Shah Pahlavi (1963)	Khouzestan, Iran	666 (203)	696 (212)
Almendra (1970)	Salmanca, Spain	662 (202)	1,860 (567)
Vidraru-Arges	Rumania	548 (167)	588 (292)
Gocekaya	Turkey	521 (159)	1,620 (494)
Morrow Point	Colorado	465	720
Pacoima	California	372 (113)	589 (180)

TABLE 13.3: COMPENSATION FLOW PUMP DATA

Pump	
Type	vertical, axial, or mixed flow
Rated head (total dynamic level)	35 ft
Rated discharge	115,000 gal/min
Pump input	1,300 hp
Speed	400 rpm
Impeller diameter	51 in (approx.)
Motor	
Type	vertical induction
Rated power	1,400 hp
Speed	400 rpm
Voltage	4,160 V
No. phases	3
Frequency	60 hz

TABLE 13.4: PRELIMINARY UNIT DATA

1 - GENERAL DATA

- Number of units	4
- Nominal unit output	150 MW
- Headwater levels	
- normal maximum	El 1445
- minimum	El 1390
- Tailwater levels	
- minimum	El 860
- normal	EL 840
- maximum	El 838

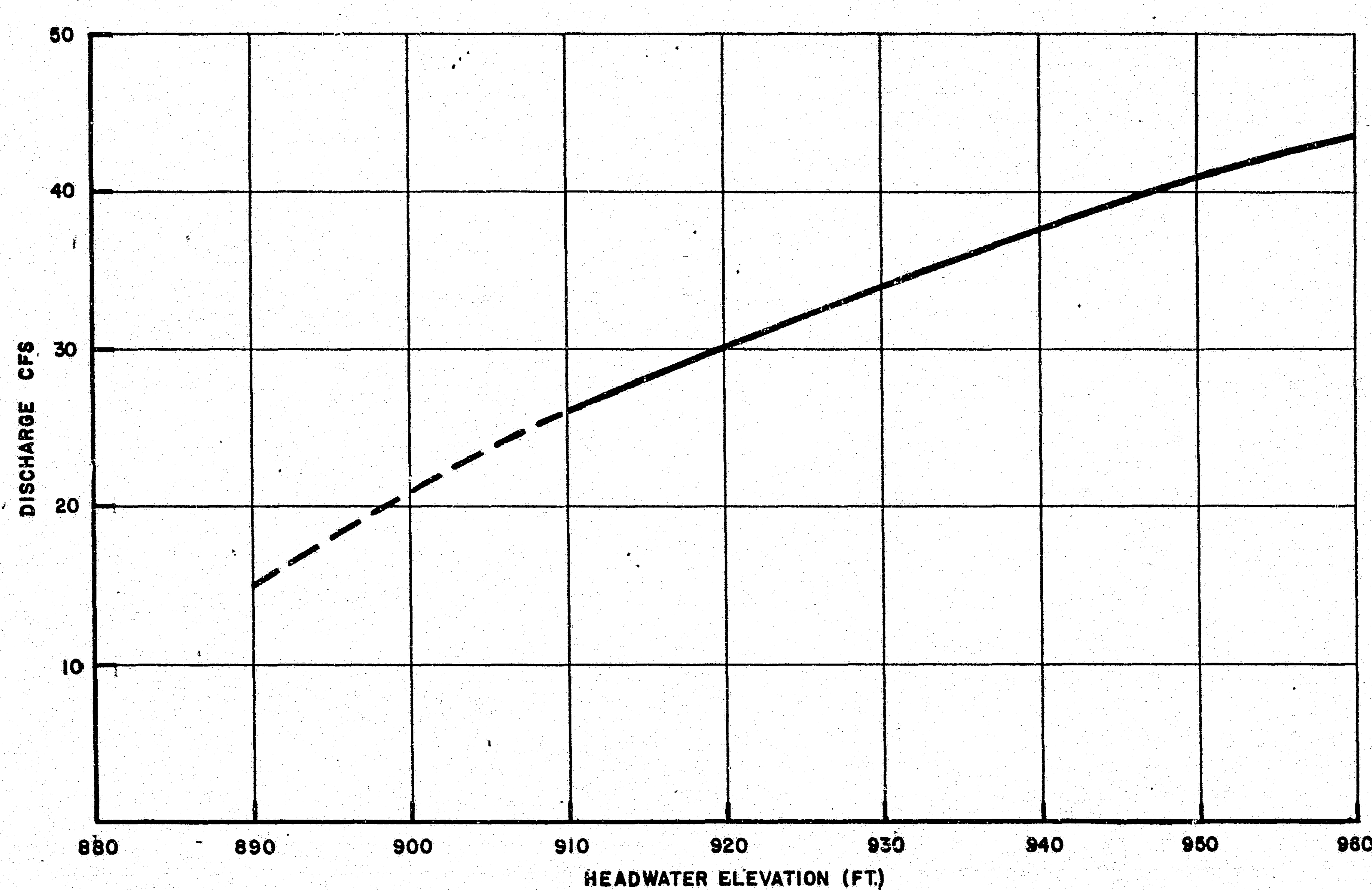
2 - TURBINE DATA

- Type	vertical Francis
- Rated net head	575 ft
- Maximum head	597 ft
- Minimum head	538 ft
- Full gate output:	
- at rated level	225,000 hp
- at maximum head	240,000 hp
- at minimum head	205,000 hp
- Best gate output	85% full gate output
- Full gate discharge at rated head	3,790 cfs
- Speed	225
- Specific speed	37.9
- Runner discharge diameter	135 in
- Runaway speed	395 rpm
- Centerline distributor	820
- Cavitation coefficient (sigma)	0.089

3 - GENERATOR DATA

- Type	vertical modified umbrella
- Rated output	180 MVA
- Power factor	0.9
- Voltage	15 kV
- Inertia constant (H)*	3.5 MW-sec/MVA
- Synchronous speed	225 rpm
- Flywheel effect (WR^2)*	54×10^6 lb-ft^2
- Heaviest lift	750,000 lb

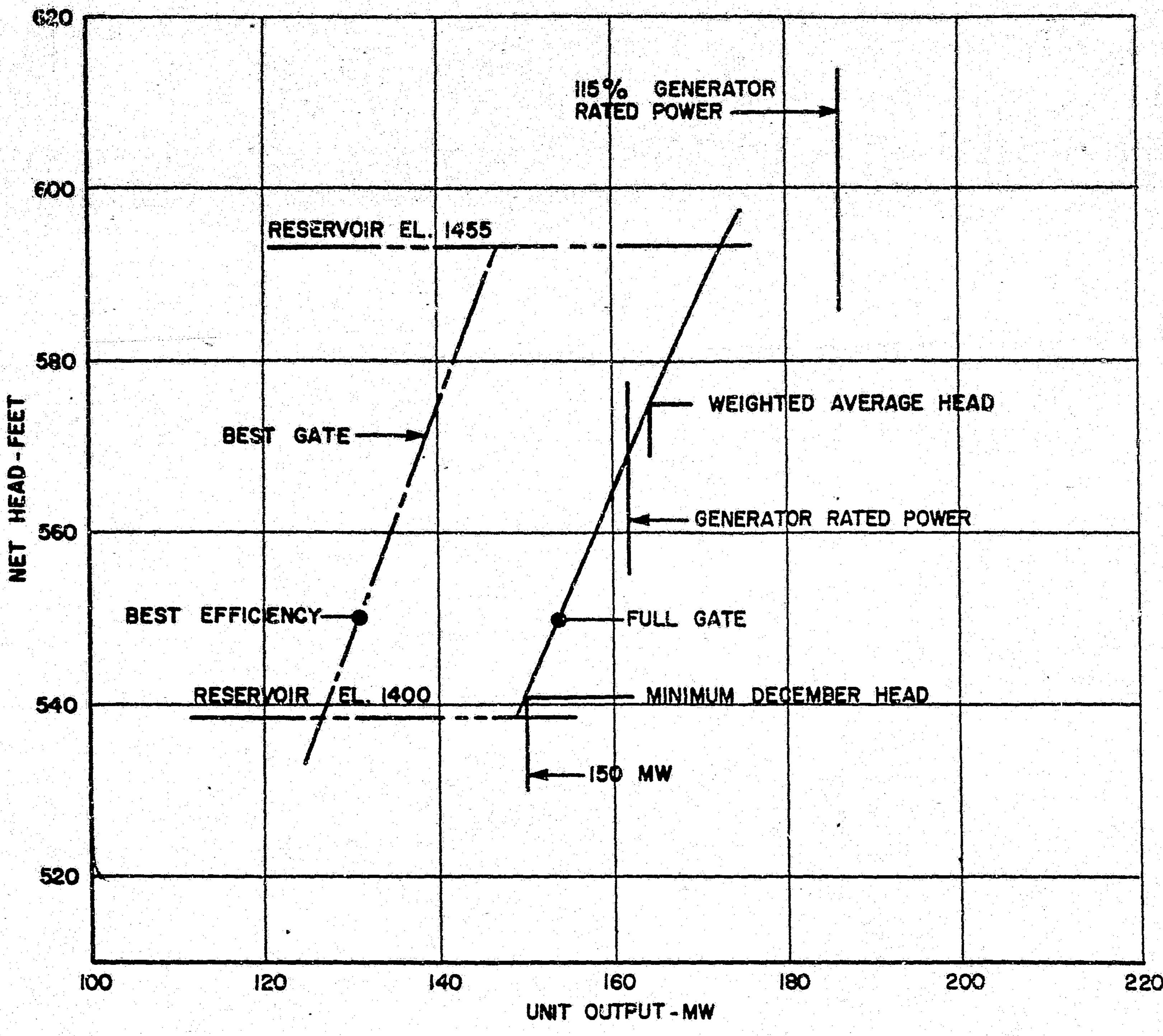
* Including turbine



DEVIL CANYON DIVERSION
RATING CURVE

ACRES

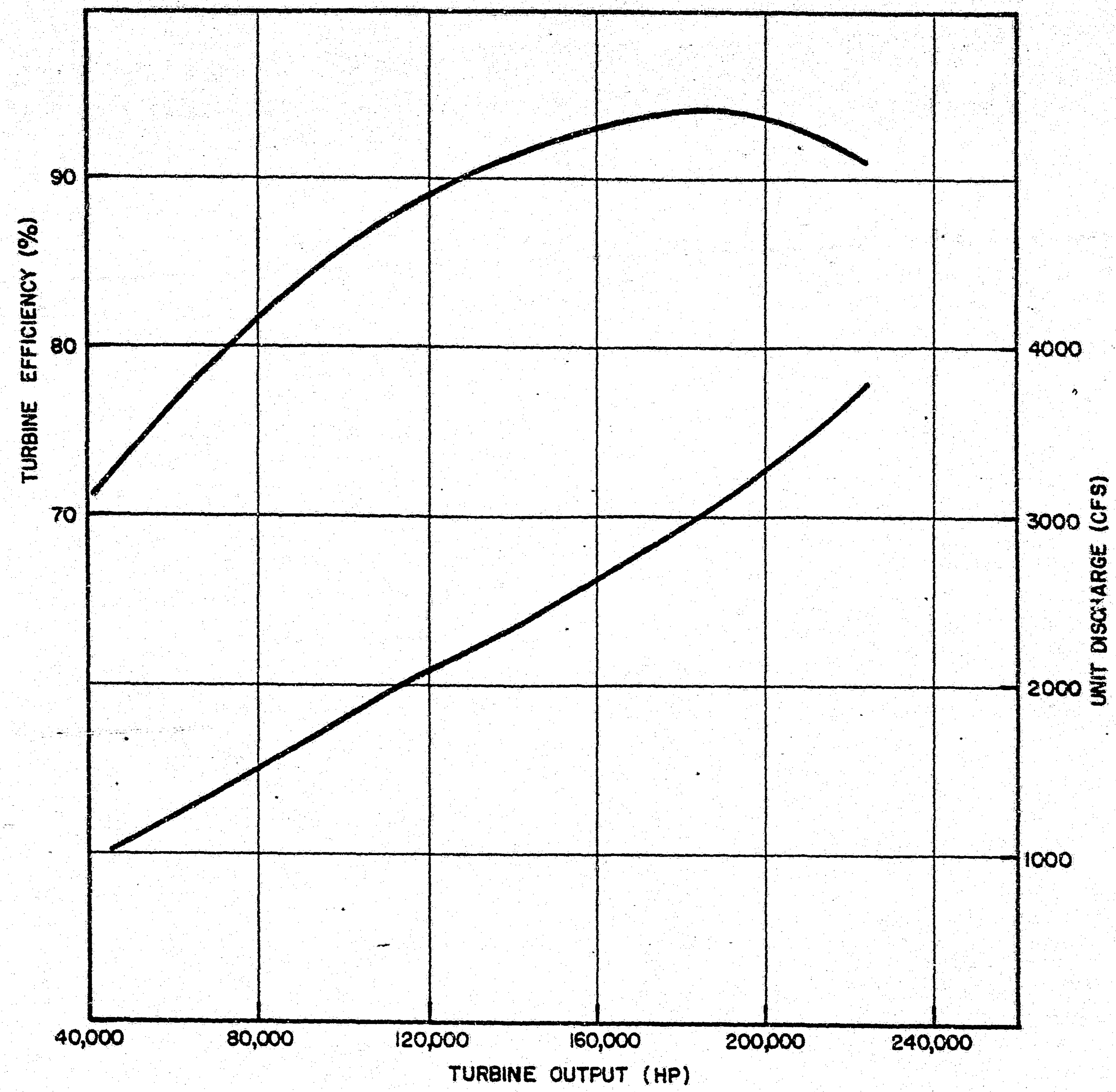
FIGURE 13.1



DEVIL CANYON - UNIT OUTPUT

FIGURE 13.2

ACRES



DEVIL CANYON - TURBINE PERFORMANCE
(AT RATED HEAD)

FIGURE 13.3

14 - TRANSMISSION FACILITIES

The objective of this section is to record and describe the studies performed to select a power delivery system from the Susitna River basin generating plans to the major load centers in Anchorage and Fairbanks. This system will be comprised of transmission lines, substations, a dispatch center, and means of communications.

The major topics of the transmission studies include:

- Electric system studies;
- Transmission corridor selection;
- Transmission detailed route selection;
- Transmission towers, hardware and conductors;
- Substations; and
- Dispatch center and communications.

Further discussion of the importance of these studies in determining the method of operation of the Railbelt System is presented in Section 15.

In the following text, each of the major topics will be discussed with respect to previous studies, methodology, additional data obtained, and conclusions arising from the studies.

14.1 - Electric System Studies

Transmission planning criteria were developed to ensure the design of a reliable and economic electrical power system, with components rated to allow a smooth transition through early project stages to the ultimate developed potential.

Strict application of optimum, long-term criteria would require the installation of equipment with ratings larger than necessary at excessive cost. In the interest of economy and long-term system performance, these criteria were temporarily relaxed during the early development stages of the project. Although allowing for satisfactory operation during early system development, final system parameters must be based on the ultimate Susitna potential.

The criteria are intended to ensure maintenance of rated power flow to Anchorage and Fairbanks during the outage of any single line or transformer element. The essential features of the criteria are:

- Total power output of Susitna to be delivered to one or two stations at Anchorage and one at Fairbanks;

- "Breaker-and-a-half" switching station arrangements;

- Overvoltages during line energizing not to exceed specified limits;

- System voltages to be within established limits during normal operation;

- Power delivered to the loads to be maintained and system voltages to be kept within established limits for system operation under emergency conditions;

- Transient stability during a 3-phase line fault cleared by breaker action with no reclosing; and

- Where performance limits are exceeded, the most cost effective corrective measures are to be taken.

(a) Existing System Data

Data have been compiled in a draft report by Commonwealth Associates Inc., dated November, 1980, and entitled "Anchorage-Fairbanks Transmission Intertie - Transmission System Data". The contents of this report have been included, with minor revisions, as Appendix B of the Susitna Hydroelectric Project Planning Memorandum - Preliminary Transmission System Analysis (1). Other system data were obtained in the form of single-line diagrams from the various utilities.

(b) Power Transfer Requirements

The Susitna transmission system must be designed to ensure the reliable transmission of power and energy generated by the Susitna Hydroelectric Project to the load centers in the Railbelt area. The power transfer requirements of this transmission system are determined by the following factors:

- System demand at the various load centers;

- Generating capabilities at the Susitna project; and

- Other generation available in the Railbelt area system.

Most of the electric load demand in the Railbelt area is located in and around two main centers: Anchorage and Fairbanks. The largest load center is Anchorage, with most of its load concentrated in the Anchorage urban area. The second largest load center is Fairbanks. Two small load centers (Willow and Healy) are located along the Susitna transmission route. The only other significant load centers in the Railbelt region are Glennallen and Valdez. However, their combined demand is expected to be less than 2 percent of the total Railbelt demand in the foreseeable future. A survey of past and present load demand levels as well as various forecasts of future trends indicates these approximate load levels at the various centers:

Load Area	Percent of Total Railbelt Load
Anchorage - Cook Inlet	78
Fairbanks - Tanana Valley	20
Glennallen - Valdez	2

Considering the geographic location and the currently projected magnitude of the total load in the area, transmission to Glennallen-Valdez is not likely to be economical in the foreseeable future. If it is ever to be economical at all, it would likely be a direct radial extension either from

Susitna or from Anchorage. In either case its relative magnitude is too small to have significant influence on either the viability or development characteristics of the Susitna project or the transmission from Susitna to the Anchorage and Fairbanks areas.

As a result, it has been assumed for study purposes that approximately 80 percent of the generation at Susitna will be transmitted to the Anchorage area and some 20 percent to Fairbanks. To account for the uncertainties in future local load growth and local generation development, the Susitna transmission system was designed to be able to transmit a maximum of 85 percent of Susitna generation to Anchorage and a maximum of 25 percent to Fairbanks.

The potential of the Susitna Hydroelectric Project is expected to be developed in three or four stages as the system load grows over the next two decades. The transmission system must be designed to serve the ultimate Susitna development, but staged to provide reliable transmission at every intermediate stage. Present plans call for three stages of Susitna development, at Watana in 1993 followed by a further 340 MW in 1997, and 600 MW at Devil Canyon in 2002.

Development of other generation resources could alter the geographic load and generation sharing in the Railbelt, depending on the location of this development. However, current studies indicate that no other very large projects are likely to be developed until the full potential of the Susitna project is utilized. The proposed transmission configuration and design should, therefore, be able to satisfy the bulk transmission requirements for at least the next two decades. The next major generation development after Susitna will then require a transmission system determined by its own magnitude and location.

The resulting power transfer requirements for the Susitna transmission system are indicated in Table 14.1.

(c) Transmission Alternatives

Because of the geographic location of the various centers, transmission from Susitna to Anchorage and Fairbanks will result in a radial system configuration. This allows significant freedom in the choice of transmission voltages, conductors, and other parameters for the two line sections, with only limited dependence between them. In the end, the advantages of standardization for the entire system will have to be compared to the benefits of optimizing each section on its own merits. Transmission alternatives were developed for each of the two system areas, including voltage levels, number of circuits required, and other parameters, to satisfy the necessary transmission requirements of each area.

Having established the peak power to be delivered and the distances over which it is to be transmitted, transmission voltages and number of circuits required were determined. To maintain a consistency with standard ANSI voltages used in other parts of the United States, the following voltages were considered for Susitna transmission:

- Watana to Devil Canyon and on to Anchorage: 500 kV or 345 kV
- Devil Canyon to Fairbanks: 345 kV or 230 kV

(i) Susitna to Anchorage

Transmission at either of two different voltage levels (345 kV or 500 kV) could reasonably provide the necessary power transfer capability over the distance of approximately 140 miles between Devil Canyon and Anchorage. The required transfer capability of 1,377 MW is 85 percent of the ultimate generating capacity of 1,620 MW. At 500 kV, two circuits would provide more than adequate capability. At 345 kV, either three circuits uncompensated or two circuits with series compensation are required to provide the necessary reliability for the single contingency outage criterion. At lower voltages an excessive number of parallel circuits are required, while above 500 kV two circuits are still needed to provide service in the event of a line outage.

(ii) Susitna to Fairbanks

Applying the same reasoning used in choosing the transmission alternatives to Anchorage, two circuits of either 230 kV or 345 kV were chosen for the section from Devil Canyon to Fairbanks. The 230 kV alternative requires series compensation to satisfy the planning criteria in case of a line outage.

(iii) Total System Alternatives

The transmission section alternatives mentioned above were combined into five realistic total system alternatives. Three of the five alternatives have different voltages for the two sections. The principal parameters of the five transmission system alternatives analyzed in detail are as follows:

	Susitna to Anchorage		Susitna to Fairbanks	
Alternative	Number of Circuits	Voltage (kV)	Number of Circuits	Voltage (kV)
1	2	345	2	345
2	3	345	2	345
3	2	345	2	230
4	3	345	2	230
5	2	500	2	230

Electric system analyses, including simulations of line energizing, load flows of normal and emergency operating conditions, and transient stability performance, were carried out to determine the technical feasibility of the various alternatives. An economic comparison of transmission system life cycle costs was carried out to evaluate the relative economic merits of each alternative. All five transmission alternatives were found to have acceptable performance characteristics. The most significant difference was that single-voltage systems (345 kV, Alternatives 1 and 2) and systems without series compensation (Alternative 2) offered reduced complexity of design and operation and therefore were likely to be marginally more reliable. The present-worth life cycle costs of Alternatives 1 through 4 were all within one percent of each other. Only the cost of the 500/230 kV scheme (Alternative 5) was 14 percent above the others. A summary of the life cycle cost analysis for the various alternatives is shown in Table 14.2. Full details of the technical and economic comparisons are explained in Reference 1.

A technical and economic comparison was also carried out to determine possible advantages and disadvantages of HVDC transmission, as compared to an ac system, for transmitting Susitna power to Anchorage and Fairbanks. As outlined in detail in Reference 1, HVDC transmission was found to be technically and operationally more complex as well as having higher life cycle costs.

(d) Configuration at Generation and Load Centers

Interconnections between generation and load centers and the transmission system were developed after reviewing the existing system configurations at both Anchorage and Fairbanks as well as the possibilities and current development plans in the Susitna, Anchorage, Fairbanks, Willow, and Healy areas.

(i) Susitna Configuration

Preliminary development plans indicated that the first project to be constructed would be Watana with an initial installed capacity of 680 MW, to be increased to approximately 1020 MW in the second development stage. The next project, and the last to be considered in this study, would be Devil Canyon, with an installed capacity of 600 MW.

(ii) Switching at Willow

Transmission from Susitna to Anchorage is facilitated by the introduction of an intermediate switching station. This has the effect of reducing line energizing overvoltages and reducing the impact of line outages on system stability. Willow is a suitable location for this intermediate switching station; in addition, it would make it possible to supply local load when this is justified by development in the area. This local load is expected to be less than 10 percent of the total Railbelt area system load, but the availability of an EHV line tap would definitely facilitate future power supply.

(iii) Switching at Healy

A switching station at Healy was considered early in the analysis, but was found to be unnecessary to satisfy the planning criteria. The predicted load at Healy is small enough to be supplied by local generation and the existing 138 kV transmission from Fairbanks.

(iv) Anchorage Configuration

In its 1975 report on the Upper Susitna River Hydroelectric Studies (2), the United States Department of the Interior Corps of Engineers favored a transmission route terminating at Point MacKenzie.

The 1979 Economic Feasibility Study Report for the Anchorage-Fairbanks Intertie by International Engineering Company, Inc. (IECO) (3) recommended one circuit from Susitna terminating at Point MacKenzie and another passing through Palmer and Eklutna substations to Anchorage along the eastern side of Knik Arm.

Analysis of system configuration, distribution of loads, and development in the Anchorage area led to the conclusion that a transformer station near Palmer would be of little benefit. Most of the major loads are concentrated in and around the urban Anchorage area at the mouth of Knik Arm. In order to reduce the length of subtransmission feeders, the transformer stations should be located as close to Anchorage as possible.

The routing of transmission into Anchorage may be chosen from the following three possible alternatives:

- Submarine Cable Crossing From Point MacKenzie to Point Woronzof

  This would require transmission through a very heavily developed area. It would also expose the cables to damage by ships' anchors, which has been the experience with existing cables, resulting in questionable transmission reliability.

- Overload Route North of Knik Arm via Palmer

  This may be most economical in terms of capital cost in spite of the long distance involved. However, approval for this route is unlikely since overhead transmission through this developed area is considered environmentally unacceptable. A longer overland route around the developed area is considered unacceptable because of the mountainous terrain.

- Submarine Cable Crossing of Knik Arm, In the Area of Lake Lorraine and Six Mile Creek

  This option, approximately parallel to the new 230 kV cable under construction for Chugach Electric Association (CEA), includes some 3 to 4 miles of submarine cable and requires a high capital cost. Since the area is upstream from the shipping lanes to the port of Anchorage it will result in a reliable transmission link, and one that does not have to cross environmentally sensitive conservation areas.

The third alternative is clearly the best of the three options. The details of this configuration are as follows:

submarine cable crossing. To reduce cable costs the crossing could be constructed with two cable circuits plus one spare phase. This option requires a switching station at the west terminal of Knik Arm. A switching station at the west terminal would clearly require increased costs and complications for construction and operation as a result of poor access. It would also require a separate location for the tap to supply MEA.

Plans are presently underway for a bridge crossing at Knik Arm for both railway and road traffic. If these plans should be realized, transmission costs and complications could be significantly reduced by routing the cables across the bridge.

(v) Fairbanks Configuration

Susitna power for the Fairbanks area is recommended to be delivered to a single EHV/138 kV transformer station located at Ester.

(e) Recommended Transmission System

The configuration of the recommended transmission system, (Alternative 2) is shown on the single-line diagram (Figure 14.1). The main characteristics of the recommended systems are summarized in Table 14.3.

14.2 - Corridor Selection

(a) Methodology

Development of the proposed Susitna project will require a transmission mechanism to deliver electric power to the Railbelt area. The pre-building of the Intertie system will result in a corridor and route for the Susitna transmission lines between Willow and Healy. Therefore, three areas require study for corridor selection: the northern area to connect Healy with Fairbanks; the central area to connect the Watana and Devil Canyon damsites with the Intertie; and the southern area to connect Willow with Anchorage.

The corridor selection methodology followed the Susitna study plan formulation and selection methodology. Previous studies, existing data, aerial reconnaissance and limited field studies formed the data base. Using the selection criteria discussed below, corridors 3 to 5 miles wide which met these criteria were selected in each of the three study areas. These corridors were then evaluated to determine which ones met the more specific screening criteria discussed below. This screening process resulted in one corridor in each area being designated as the recommended corridor for the transmission line. For a more detailed discussion of study methodology and the selection and screening criteria, refer to the Transmission Line Corridor Screening Closeout Report (September, 1981), hereafter referred to as the Closeout Report.

(b) Previous Studies

The two reports reviewed which contained the most information relevant to the transmission line studies were:

- The Susitna Hydroelectric Project Interim Feasibility Report, prepared by the U.S. Army Corps of Engineers (hereafter referred to as the COE report); and

- The Economic Feasibility Study for the Anchorage-Fairbanks Intertie by International Engineering Company, Inc./Robert W. Retherford Associates (hereafter referred to as the IECO/RWRA report).

The COE report consisted primarily of an evaluation of alternative corridor locations to aid in the selection of those which maximized reliability and minimized costs. Utilizing aerial photographs and existing maps, general corridors connecting the project site with Anchorage and Fairbanks were selected. This study was general in nature and was intended only to demonstrate project feasibility.

The IECO/RWRA report utilized the COE report as background information for both economic feasibility determination and route selection. The corridor selected by IECO/RWRA was very similar to that selected by the COE with further definition. The route selected was based on length, accessibility and environmental compatibility. The report also presented a detailed economic feasibility study for the Anchorage-Fairbanks transmission study.

(c) Selection Criteria and Selection Results

(i) Criteria

The objective of the corridor selection conducted by Acres was to select feasible transmission line corridors in each of the three study areas:

- The northern area, to connect Healy with Fairbanks;

- The central area, to connect the Watana and Devil Canyon damsites with the Intertie; and

- The southern area, to connect Willow with Anchorage.

Technical, economic, and environmental criteria were developed in order to select corridors within the three areas. These criteria are listed in Table 14.4.

Environmental inventory tables were then compiled for each corridor selected, listing length, number of road crossings, number of river and creek crossings, topography, soils, land ownership/status, existing and proposed development, existing rights-of-way, scenic quality/recreation, cultural resources, vegetation, fish, birds, furbearers, and big game. These tables are included in the Closeout Report.

(ii) Results

Utilizing existing information, 22 corridors were selected based on their ability to meet technical, economic and environmental criteria as listed in Table 14.4.

Four of the corridors are in the southern study area, 15 in the central area, and four in the northern study area. Three of the corridors in the southern study area run in a north-south direction while one runs northeast to Palmer, then northwest to Willow. Corridors in the central study area are in two general groups: those running from Watana Damsite west to the proposed Intertie and those running north across the Denali Highway and the Chulitna River. Corridors in the northern study area run either west or east to bypass the Alaskan Range, then proceed north to Fairbanks.

See Figures 14.2, 14.3, and 14.4 for the location of these corridors.

(d) Screening Criteria and Screening Results

(i) Criteria

The objective of the screening process was to screen the previously selected corridors to determine which best meet additional technical, economic, and environmental criteria as listed in Table 14.5. The rationale for selection of these criteria is explained in Appendix E2.

In addition to these criteria, each corridor was screened for reliability to determine if a line could be operated with a minimum of power operation. Six basic factors were considered in relation to reliability:

- Elevation: Lines located at elevations below 4000 will be less exposed to severe wind and ice conditions which can interrupt service.

- Aircraft: Avoidance of areas near aircraft landing and takeoff operations will minimize the risk of collisions.

- Stability: Avoidance of areas susceptible to land, ice, and snow slides will reduce the chance of power failures.

- Existing Power Lines: Avoiding existing transmission lines will reduce the possibility of lines touching during failures and will facilitate repairs.

- Topography: Lines located in areas with gentle relief will be easier to construct and repair.

- Access: Lines located in reasonable proximity to transportation corridors will be more quickly accessible and, therefore, more quickly repaired if any failures occur.

The screening criteria and reliability factors for each corridor were evaluated utilizing topographic maps, aerial photos, aerial overflights, and published materials. Each corridor was then assigned four ratings (one each for technical, economic and environmental considerations, and one overall summary rating.) Ratings were defined as follows:

A - recommended
C - acceptable but not preferred
F - unacceptable

From the technical point of view, reliability was the main objective. An environmentally and economically sound corridor was rejected if a line built in the corridor would be unreliable. Thus, any line which received an F technical rating was assigned a summary rating of F and eliminated from further consideration.

Similarly, because of the critical importance of environmental considerations, any corridor which received an F rating for environmental impacts was assigned a summary rating of F, and eliminated from consideration.

(ii) Results

Table 14.6 summarizes the comparison of the corridor screened in the southern, central and northern study areas. One corridor in each of the three study areas received A ratings for all three categories. These three corridors and the rationale for their A ratings are discussed below. For a description of all 22 corridors and the rationale for their ratings, see Appendix E2.

- Southern Study Area

Corridor Two - Willow to Point MacKenzie via Red Shirt Lake

. Description

Corridor ADFC, consisting of Segments ADF and FC (Figure 14.2), commences at the point of intersection with the Intertie in the vicinity of Willow but immediately turns to the southwest, first crossing the railroad, then the Parks Highway, then Willow Creek just west of Willow. The land in the vicinity of this part of the segment is very flat, with wetlands dominating the terrain.

Southwest of Florence Lake, the proposed corridor turns, crosses Rolly Creek, and heads nearly due south, passing through extensive wetlands west and south of Red Shirt Lake. The corridor in this area parallels existing tractor trails and crosses very flat lands with significant amounts of tall-growing vegetation in the better drained locations.

Northwest of Yohn Lake, the corridor segment turns to the southeast, passing Yohn Lake and My Lake before crossing the Little Susitna River. Just south of My Lake, the corridor turns in a southern direction, passing Middle Lake and east of Horseshoe Lake before finally intersecting the existing Beluga 230 kV transmission line at a spot just north of MacKenzie Point. From here, the corridor parallels MacKenzie Point's existing transmission facilities before crossing under Knik Arm to emerge on the eastern shore of Knik Arm in the vicinity of Anchorage. The land in the vicinity of this segment is extremely flat and wet. It supports stands of tall-growing vegetation on the higher or better drained areas.

. Technical and Economical Rating

Corridor ADFC crosses the fewest number of rivers and roads in the southern study area. It has the advantage of paralleling an existing tractor trail for a good portion of its length, thereby reducing the need for new access roads. Easy access will allow maintenance and repairs to be carried out in minimal time. This corridor also occurs at low elevations and is approximately one-half the length of Corridor One.

. Environmental Rating

This corridor crosses extensive wetlands from Willow to Point MacKenzie. At higher elevations or in the better drained sites, extensive forest cover is encountered. Good agricultural soils have been identified in the vicinity of this corridor; the state plans an Agricultural Lands Sale for areas to be traversed by this corridor. The corridor also crosses the Susitna Flats Game Refuge. The presence of an existing tractor trail near considerable portions of this corridor diminishes the significance of some of these constraints. Furthermore, its short length and the fact that it crosses only one river and eight creeks increases its environmental acceptability.

- Central Study Area

Corridor One - Watana to Intertie via South Shore, Susitna River

. Description

This corridor originates at the Watana Dam site and follows the southern boundary of the river at an elevation of approximately 2,000 feet from Watana to Devil Canyon (Figure 14.3). From Devil Canyon, the corridor continues along the southern shore of the Susitna River at an elevation of about 1,400 feet to where it connects with the Intertie, assuming the Intertie follows the railroad corridor. The land surface in this area is relatively flat, though incised at a number of locations by tributaries to the Susitna River. The relatively flat hills are covered by discontinuous stands of dense, tall-growing vegetation.

. Technical and Economical Rating

Corridor One is one of the shortest corridors considered. It is approximately 40 miles long, making it economically favorable. No technical restrictions were observed along the entire length of this corridor.

. Environmental Rating

Because of its short length, environmental disturbance caused by transmission line construction would be reduced. The more noteworthy constraints are those identified under the categories of land use and vegetation. Corridor One would require the development of a new right-of-way between Watana and Devil Canyon with some opportunity existing to utilize the COE-developed road for access between the Intertie and Devil Canyon. The potential does exist in this corridor to use the proposed access road. Wetlands and discontinuous forest cover occur in the corridor, especially in the eastern third of the route. Access road development, and the associated vegetation clearing, present additional constraints to this corridor.

- Northern Study Area

Corridor One - Healy to Fairbanks via Parks Highway

. Description

Corridor One (ABC), consisting of Segments AB and BC, starts in the vicinity of the Healy Power Plant (Figure 14.4). From here, the corridor heads northwest, crossing the existing Golden Valley Electric Association Transmission Line, the railroad, and the Parks Highway before turning to the north and paralleling this road to a point due west of Browne. Here, as a result of terrain features, the corridor turns northeast, crossing the Parks Highway once again as well as the existing transmission line, the Nenana River, and the railroad, and continues to a point northeast of the Clear Missile Early Warning Station (MEWS).

Continuing northward, the corridor eventually crosses the Tanana River east of Nenana, then heads northeast, first crossing Little Goldstream Creek, then the Parks Highway just north of the Bonanza Creek Experimental Forest. Before reaching the drainage of Ohio Creek, this corridor turns back to the northeast, crossing the old Parks Highway and heading into the Ester Substation west of Fairbanks.

Terrain along this entire corridor segment is relatively flat, with the exception of the foothills north of the Tanana River. Much of the route, especially that portion between the Nenana and the Tanana River crossings, is very broad and flat. It has standing water during the summer months and, in some places, is overgrown by dense stands of tall-growing vegetation. This corridor segment crosses the heavily wooded foothills northeast of Nenana.

An option to the above not shown in the figures has been considered, closely paralleling and sharing rights-of-way with the existing Healy-Fairbanks transmission line. While it is usually attractive to parallel existing corridors wherever possible, this option necessitates a great number of road crossings and results in an extended length of the corridor paralleling the Parks Highway. A potentially significant amount of highway-abutting land would be usurped for containment of the right-of-way. The combination of these features precludes this corridor from further evaluation.

. Technical and Economical Rating

This corridor crosses the fewest water courses in the northern study area. Although it is approximately four miles longer than Corridor Two, it is technically favored because of the existence of potential access roads for almost the entire length.

. Environmental Rating

Because it parallels an existing transportation corridor for much of its length, this corridor would permit line routing that would avoid most visually sensitive areas. The three proposed road crossings for this corridor (as opposed to the 19 road crossings of the Healy-Fairbanks transmission line) could occur at points where roadside development exists, in areas of visual absorption capability, or in areas recommended to be opened to long-distance views.

Four rivers and 40 creeks with potential for impacts are crossed by this corridor. It crosses the fewest number of water courses of any route under consideration in the northern study area. In addition, the inactive nest site of a pair of peregrine falcons occurs within this proposed corridor.

As with visual impacts, land use, wildlife, and fishery resource impacts can be lessened through carefully route location and utilization of existing access. Impacts on forest clearing can be lessened through the sharing of existing transmission line corridors.

(e) Conclusions

A review of previous reports, other existing information, and aerial overflights was used to select corridors for consideration in this study. These corridors were screened against certain technical, economic and environmental criteria, resulting in one recommended corridor in each of the southern, central and northern study areas. The corridors shown in Figures 7.1 through 7.8 of Appendix E2 are believed to best meet the technical, economic and environmental criteria; therefore, these corridors are the best locations in which to place the Susitna transmission lines.

14.3 - Route Selection

(a) Methodology

After identifying the preferred transmission line corridors, the next step in the route selection process involved the analysis of the data as gathered and presented on the base map. Overlays were compiled so that various constraints affecting construction or maintenance of a transmission facility could be viewed on a single map. The map is used to select possible routes within each of the three selected corridors. By placing all major constraints (e.g., area of high visual exposure, private lands, endangered species, etc.) on one map, a route of least impact was selected. Existing facilities, such as transmission lines and tractor trails within the study area, were also considered during the selection of a least impacted route. Whenever possible, the routes were selected near existing or proposed access roads, sharing whenever possible existing rights-of-way.

The data base used in this analysis was obtained from the following sources:

- An up-to-date land status study;
- Existing aerial photos;
- New aerial photos conducted for selected sections of the previously recommended transmission line corridors;
- Environmental studies including aesthetic considerations;
- Climatological studies;
- Geotechnical exploration;
- Additional field studies; and
- Public opinions.

(b) Selection Criteria

(i) Criteria

The purpose of this section is to identify three selected routes: one from Healy to Fairbanks, the second from Watana-Devil Canyon damsites to the intertie, and the third from Willow to Anchorage.

The previously chosen corridors were subject to a process of refining and evaluation based on the same technical, economic, and environmental criteria used in corridor selection (see Table 14.15). In addition, special emphasis was concentrated on the following points:

- satisfy the regulatory and permit requirements;
- selection of routing that provides for minimum visibility from highways and homes; and
- avoidance of developed agricultural lands and dwellings.

(ii) Results

Figures 1 through 14 in Appendix E3 show the selected transmission line route for the three areas of study; namely, the southern study area; the central study area; and the northern study area. As a first step, the 3-to-5-mile-width corridor previously selected for each of the three study areas was narrowed to a half-mile-width corridor based on the previous criteria. The preliminary centerline of the right-of-way is shown in the figures. This centerline represents a right-of-way width of 400 feet. This width is adequate for three, single-circuit, parallel lines with tower structures having horizontal phase spacing of 33 feet. However, between the Devil Canyon damsite and the intertie, the width of the right-of-way is 700 feet which is needed to accommodate five single-circuit lines.

Southern Study Area - Willow to Point Mackenzie
Via Knik Arm Crossing

Description

Evironmental Considerations

Technical and Economic Considerations

This route crosses a very few number of rivers and roads. It parallels the existing tractor trail for a considerable portion of its length, thereby reducing the need for new access roads.

At Knik Arm, the route parallels a proposed 230 kV transmission line and will share (if possible) its right-of-way. This will avoid pioneering new right-of-way especially in an area full of dwellings and other constraints.

Central Study Area - Watana to Intertie

Description

Environmental Considerations

Technical and Economic Considerations

This is the shortest route among the studied ones in the central study area, which makes it economically feasible. The route parallels a proposed access road almost through its entire length north of Susitna River for the section between the two damsites and south of it between Devil Canyon and the Intertie. This will add to the reliability and economical considerations.

Northern Study Area - Healy to Ester

Description

Environmental Considerations

Technical and Economic Considerations

The existence of access roads for almost the entire length makes this selected route technically and economically feasible. Its short length will add to its economical considerations.

(c) Route Soil Conditions

(i) Description

Baseline geological and geotechnical information has been compiled through photointerpretation and terrain unit mapping (Reference 4). The general objective was to document the conditions that would significantly affect the design and construction of the transmission line towers. More specifically, the objectives included the delineation of and forms of various origins, noting the occurrence and distribution of significant geologic factors such as permafrost, potentially unstable slopes, potentially erodible soils, possible active fault traces, potential construction materials, active floodplains, organic materials, etc.

Work on the air photointerpretation consisted of several activities culminating in a set of terrain unit maps delineating surface materials and geologic features and conditions in the project area.

The first activity consisted of a review of the literature concerning the geology of the Intertie corridors and transfer of the information gained to high-level photographs at a scale of 1:63,000. Interpretation of the high-level photos created a regional terrain framework which helped in the interpretation of the low-level 1:30,000 project photos. Major terrain divisions identified on the high-level photos were then used as an aerial guide for delineation of more detailed terrain units on the low-level photos. The primary effort of the work was the interpretation of 140-plus photos covering about 300 square miles of varied terrain. The land area covered in the mapping exercise is shown on map sheets and displayed in detail on photo mosaics.

As part of the terrain analysis, the various bedrock units and dominant lithologies were identified using published U.S. Geological Survey reports. The extent of these units was roughly delineated on the photographs, and using exposure patterns, shade, texture, and other features of the rock unit as they appeared on the photographs, unit boundaries were drawn. Terrain unit symbols denoting the various lithologies were utilized on the maps.

Physical characteristics and typical engineering properties of each terrain unit were considered and a large chart for each corridor was developed. The charts identify the terrain units as they have been mapped and characterize their properties in numerous categories. This allows an assessment of each unit's influence on various project features.

(ii) Terrain Unit Analysis

The terrain unit is a special purpose term comprising the land forms expected to occur from the ground surface to a depth of about 25 feet.

The terrain unit maps for the proposed Anchorage to Fairbanks transmission line show the aerial extent of the specific terrain units which were identified during the air photo investigation and were corroborated in part by a limited onsite surface investigation. The units document the general geology and geotechnical characteristics of the area.

The north and south corridors are separated by several hundred miles and not surprisingly encounter different geomorphic provinces and climatic conditions. Hence, while there are many landforms (or individual terrain units) that are common to both corridors, there are also some landforms mapped in just one corridor. The landforms or individual terrain units mapped in both corridors were briefly described.

Several of the landforms have not been mapped independently but rather as compound or complex terrain units. Compound terrain units result when one landform overlies a second recognized unit at a shallow depth (less than 25 feet), such as a thin sheet of glacial till overlying bedrock or a mantle of lacustrine sediments overlying till. Complex terrain units have been mapped where the surficial exposure pattern of two landforms are so intricately related that they must be mapped as a terrain unit complex, such as some areas of bedrock and colluvium. The compound and complex terrain units were described as a composite of individual landforms comprising them. The stratigraphy, topographic position, and aerial extent of all units, as they appear in each corridor, were summarized on the terrain unit properties and engineering interpretations chart.

(d) Conclusions

A study of existing information and aerial overflights, together with additional aerial coverage, was used to locate the recommended route in each of the southern, central, and northern study areas.

Additional environmental information and land status studies made it possible to align the routes to avoid any restraints.

Terrain unit maps describing the general material expected in the area were prepared specifically for transmission line studies and were used to locate the routes away from unfavorable soil conditions whenever possible.

The route shown in Figure 14.5, represents the general location of the recommended route. It also shows the existing surface transportations.

Figures 1 through 14 of Appendix E3 are believed to best meet the technical, economical, and environmental criteria. In these figures, a half-mile-wide corridor is located; within this width, the centerline of the proposed route is located. The centerline represents 400-foot-width right-of-way for the entire length of the transmission line, except in the segment of the line between Devil Canyon and the Intertie where the right-of-way is 700 feet. This segment has five single-circuit lines parallel to each other.

14.4 - Towers, Foundations and Conductors

A transmission line intertie between Anchorage and Fairbanks is planned by APA. The intertie will consist of existing lines and a new section between Willow and Healy. The new section will be built to 345 kV standards and will be fully compatible with Susitna requirements.

(a) Transmission Line Structures

(i) Selection of Tower Type

Because of unique soil conditions in Alaska, with extensive regions of muskey and permafrost, conventional self-supporting or rigid towers will not provide a satisfactory performance or solution for the proposed transmission line.

Permafrost and seasonal changes in the soil are known to cause large earth movements at some locations, requiring towers with a high degree of flexibility and capability for handling relatively large foundation movements without appreciable loss of structural integrity.

The guyed tower is exceptionally suitable for these types of conditions. The recommended type of structure for this study is therefore the hinged-guyed steel x-tower (Figure 14.6).

The design features include hinged connections between the leg members and the foundations which, together with the longitudinal guy system, provide for large flexibility combined with excellent stability in the direction of the line. Transverse stability is provided by the wide leg base which also accounts for relatively small and manageable footing reactions.

In addition to the above features of the selected structure, the following are important favorable points:

- The x-frame structure provides for less visual impacts than other structures. This results from the simplicity of design;

- It requires little maintenance except for guy adjustment when needed;

- Towers could be stored in remote areas without concern for replacement when needed because of vandalism or deterioration;

- Easy for construction, a typical tangent structure consists of only six major components with bolted connections. All bolts are loaded in shear to eliminate any special consideration for torquing of assembly nuts during field assembly; and

- Economically attractive, especially when considering all the engineering features it combines.

The tangent tower shown in the figure represents the majority of structures required for the transmission line (about 90 percent of the total structures). However, special types of structures may be needed to satisfy a specific field requirement, wire stringing or line angles.

Also, it is anticipated that the section between Knik Arm and University Substation will consist of double-circuit, self-supporting, single-pole type structures.

(ii) Design Parameters

- Clearance to Ground: A ground clearance of 32 feet may be adequate against expected field tolerances.

- Spans: A wind span of 1400 feet, and weight span of 1600 feet is recommended for the tangent structure. This will combine economy and flexibility in spotting the structures. Longer spans, however, may be needed to overcome specific site location.

- Structure Height: An average height of 85 feet is expected throughout the transmission lines.

- Insulations: The center phase is a V-string assembly to control swing and to provide for smaller phase spacing. The side strings are of the single type.

- Shield Wires: The shield wire when needed will be 3/8 inch by 7 strand, EHS steel.

(iii) Loading Conditions

Climatic studies for transmission lines were performed to determine likely wind and ice loads based on historical data. A more detailed study incorporating additional climatic data was performed to confirm or modify the obtained data.

Details of the climatic studies for transmission lines may be found in Appendix B-6.

The design loads acting on wires and structures are mainly based upon weather conditions. Four cases of loadings are thus established for the tower design. These are:

- National Electrical Safety Code (NESC) Heavy Loading: This consists of 1/2-inch radial ice around the wires, 40 miles per hour wind at 0°F. This wind speed produces four pounds per square inch pressure acting on the project areas of cylindrical surfaces.

- Extreme Wind Loading: Consists of 140-mph wind (produces 50 psf on cylindrical surfaces), no ice at 32°F.

- Heavy Ice: Consists of one-inch ice with no wind at 32°F.

- Longitudinal Pull: No ice, no wind, 32°F. The pull pressure should be applied at any one conductor attachment.

The first case, NESC heaving loadings, will be applied to the major part of the transmission line; however, in certain areas where the weather conditions are more severe, the second and third case may be applied.

Loadings Upon Supporting Structures

- Description of Loads

  . Vertical loads on supporting structures and foundations shall be their own weight plus the superimposed weight which they support, including all wires, ice coated when specified. The effective vertical span for wires shall be determined with proper consideration of the effect of support at different elevations. The weight of ice shall be determined based on 57 pounds per cubic foot.

  . Transverse Loading (perpendicular to the wires). This shall be determined from the following conditions: wind in conjunction with ice and wind without ice. The effective span of determining the wind on wires shall be equal to one-half the sum of the adjacent spans between supporting structures.

Where a change in direction of wires occurs, a transverse loading upon the supporting structure shall be a resultant load equal to the vector sum of the maximum transverse wind load and the resultant load imposed by the wires because of their change in direction. In obtaining these loadings, a wind direction shall be used which will give the maximum resultant load.

- Longitudinal Loading

Proper allowance should be made for longitudinal loads which may be produced on structures by wire stringing operations and construction techniques.

- Application of Loads

The vertical, transverse, and longitudinal loadings previously specified relate to loadings on the wires and structures. The component of these loadings should be considered acting simultaneously.

(iv) Results

The hinged-guyed steel x-type tower is selected as the basic structure for the project because of its flexibility and performance in withstanding the unique soil conditions in Alaska. Also, the x-tower is rated very favorably concerning reliability, maintenance, construction, economy, and aesthetics.

(b) Tower Foundations

(i) Geological Conditions

A generalized terrain analysis was conducted to collect geologic and geotechnical materials data for the transmission line corridors, a relatively large area. The engineering characteristics of the terrain units have been generalized and described qualitatively. When evaluating the suitability of a terrain unit for a specific use, the actual properties of that unit should be verified by onsite subsurface investigation, sampling, and laboratory testing.

The three main types of materials along the transmission line are:

- Good material, which is defined as material which permits augered excavation and allows installation of concrete without special form work;

- Wetland and permafrost material which requires additional design details providing additional depth: and

- Rock material is defined as material in which drilled-in anchors and concrete footings can be used.

Based on aerial, topographic, and terrain unit maps, the following is recorded:

- For the southern study area: Wetland and permafrost materials constitute the major part of this area. Some rock and good foundation materials are present in this area in a very small proportion.

- For the central study area: Rock foundation and good materials were observed in most of this study area.

- For the northern study area: The major part in this area is the wetland and permafrost materials. Some parts have good rock materials.

(ii) Types of Foundations

The recommended two-legged x-frame tower is hinged at the foundation attachment connection for longitudinal freedom and restrained by fore and aft guying to an equalizing yoke. This arrangement will result in relatively smaller loads on the foundations. The recommended types of foundations are shown in Figure 14.7. These are:

- Rock Anchor

  This type of footing is specified whenever good quality rock is encountered near the ground surface. The concrete piers are grouted into the rock with reinforcing bars; permissible bearing values with this type of footing are high. The entire hole, as shown in the figure, can be drilled using the small diameter hole size without casing. This type of hole is easy and quick to drill and presents little or no problems.

  The minimum depth of these holes is approximately eight feet, with the entire depth grouted to insure adequate anchoring below the maximum frost depth.

- Pile Foundation

  Most of the transmission line towers will be supported by pile-type foundations consisting of heavy H-pile beams driven to variable depths, depending upon the soil conditions.

  This type of footing is considered in the three study areas when a good bearing stratum does not occur at normal footing depth or at a reasonable distance below. The piles driven to firm strator until the required penetration is reached would be less costly than other types of footings for the same type of soil. A minimum safety factor of 2 is recommended to be applied against uplift. The uplift resistance is always dependent on the skin friction between pile and soil. The safety factor may be varied if soil tests or pile loading tests indicate that uplift resistance of the pile is greater or less.

(iii) Design Criteria

The greater part of the combined maximum reactions on transmission structure footing is usually from temporary loads such as broken wire, wind, and ice. With the exception of heavy-angle, dead-end, or terminal structures, only a part of the total reaction is of a permanent nature. As a consequence, the permissible soil pressure as used in the design of building foundations may be considerably exceeded for footing for transmission structures.

The permissible values of soil pressure used in the footing design will depend on the structure and the supporting soil. The basic criterion is that displacement of the footing is not restricted because of the flexibility of the selected x-frame structure and the hinged connection to the tower footing. The shape and configuration of the selected tower are important factors in foundation considerations.

Loads on the tower consist of vertical and horizontal loads and are transmitted down to the foundation and then distributed to the soil. In a tower placed at an angle or used as a dead-end in the line, the horizontal loads are responsible for a large portion of the loads on the foundation. In addition to the horizontal shear, a movement is also present at the top of the foundation, creating vertical down-load and uplift forces on the footing.

To select and detail design the most economical type of foundation for a specific tower location, soil conditions at the site must be known. Soil investigation will furnish this needed information. A soil boring is a guide to the type of soil and its strength in resisting the forces on the tower. The cost of soil borings is small compared to the line cost per mile. The primary purpose of soil borings is to assure an adequate and safe foundation.

(c) Conductor Requirements

(i) Conductor Size

Based on the transmission and power transfer requirements at the various stages of the Susitna development, economic conductor sizes were determined.

The methodology used to obtain the economic conductor size and the results obtained are outlined in Appendix E1. Also included in the Appendix are the capitalized costs of transmission line losses.

When determining appropriate conductor size, the economic conductor is checked for radio interference (RI) and corona performance. If RI and corona performance are within acceptable limits, then the economic conductor size is used. However, where the RI and corona performance are found to be limiting, the conductor selection is based on these requirements.

(ii) Recommended Sizes

The Susitna transmission has been divided into three geographical sections each of which has particular loading and environmental requirements.

In the section from Devil Canyon to Willow to Knik Arm, there will be three circuits ultimately. Line loadings are sufficiently heavy to have a significant effect on conductor selection, and the economic choice is considered to be:

- 2 x 954 kcmil, 45/7 Aluminum Conductor Steel Reinforced (ACSR), "Rail" conductor. This conductor will also be used on the two circuits between Watana and Devil Canyon.

In the section from Knik Arm to University Substation, the line loading is also heavy, but environmental considerations dictate the use of two circuits instead of three and the economic conductor size is:

- 2 x 1,351 kcmil, 54/19 ACSR, "Martin" conductor.

In the northern section from Devil Canyon to Fairbanks, line loadings are light and the economic conductor size would be smaller than that allowed by RI and corona considerations. In this section the minimum conductor that can be used is:

- 2 x 795 kcmil, 26/7 ACSR, "Drake" conductor.

However, since the intertie between Willow and Healy will be prebuilt with 2 x 954 kcmil, it may be advantageous to standardize by constructing all of the remaining Susitna transmission to Fairbanks with 2 x 954 kcmil. This could be determined later.

14.5 - Substations

To follow

14.6 - Dispatch Center and Communications

(a) Existing Railbelt Dispatching Facilities

The main generating and load centers are located in Fairbanks and Anchorage. Both areas operate independently of each other. It is proposed by APA to connect the two systems by a tie in 1984 which will be operated at 138 kV. The power transfer capability will be approximately 70 MW.

Golden Valley Electric Association (GVEA) and Fairbanks Municipal Utility System (FMUS) constitute the two major producers of electrical power in Fairbanks. Although both utilities are intertied at 69 kV, they each provide their own dispatching. GVEA is responsible for maintaining frequency in the Fairbanks area.

Chugach Electric Association (CEA), Anchorage Municipal Light and Power (AMLP) and Matanuska Electric Association (MEA) are the utilities which serve the Anchorage area and all areas north of Anchorage, including Willow and Palmer. MEA is mainly a distributing utility and imports its power from CEA and the Alaska Power Administration. AMLP generates and distributes power in the Anchorage area. CEA generates and distributes electric power in Anchorage and adjoining areas. Each utility provides its own control center facilities and is intertied at the 115 and 138 kV level.

Presently, CEA provides frequency control for the Anchorage area. CEA has its own system control center that provides dispatching and supervisory functions over its generating and substation facilities.

(b) 1993 Railbelt Power System

The introduction of Susitna hydroelectric power in the Railbelt area will require several hundred miles of transmission lines from the Susitna River basin to Anchorage and Fairbanks. In fact, the ultimate development will require approximately 850 miles of transmission, 5 switchyards and 2 hydro generating stations at Watana and Devil Canyon. Thermal generation at Fairbanks and Anchorage will still be in operation. The installed generation capacity will be over 2,000 MW at that time.

To operate such an enlarged Railbelt system, a control system or energy management system (EMS) will be required. This system will insure security of the 345 kV transmission lines and switchyards/substations operations. The system will also exercise remote control and efficient dispatching of the generating units in the Railbelt.

(c) Energy Management System Requirements

To provide an efficient and secure dispatching system for the Railbelt, the following functions are proposed:

(i) Supervisory Control and Data Acquisition (SCADA) Subsystem

Includes real-time system data acquisition; remote control of power system devices; data base and data base management; data processing; operation data logging and report generation; and man/machine interface requirements.

(ii) Generation Control Subsystem

Includes automatic control of hydro and thermal units in the Railbelt area to maintain interconnected system frequency and interchange scheduling; economic unit operation; generation reserve evaluation; and monitoring of system generation performance.

(iii) Power Scheduling and Load Forecasting Subsystem

Includes the forecasting of system load and the scheduling of the power system generation to meet load requirements in the most economical and reliable way.

(iv) Energy Accounting Subsystem

Includes collection, recording, and processing of data power transactions among various utilities in the interconnected system; also the cost information and the savings/losses resulting from purchase/sale of power.

(v) System Security Subsystem

Includes the ability to evaluate system performance based on present and predicted system conditions and the ability to evaluate the impact of probable contingencies (loss of generation, loss of a transmission line, etc.).

(vi) System Support Subsystem

Includes on-line/off-line functions that could be performed by EMS to support engineering, accounting, and system operation organizations.

A more detailed description of the functional requirements is covered by a report entitled, "Energy Management System (EMS) - System Requirements" dated December, 1981.

(d) Energy Management System Alternatives

An evaluation of alternative system configurations showed that two different approaches to generation control are possible:

- Alternative I provides indirect control of generating units; and
- Alternative II provides direct control of generating units.

To formulate and evaluate the alternative configuration, the following criteria were used:

- Configurations must fulfill functional requirements discussed above in paragraph (c);

- Configurations must be technically, economically, and operationally maintainable through the life of the systems (10 to 15 years); and

- Configuration must be technically feasible, as well as proven.

(i) Alternative I System Configuration

The Alternative I system configuration is typical of the current offerings of several EMS equipment manufacturers (see Figure 14.12, EMS Alternative I System Configuration). The configuration is based on the assumptions that:

- An in-plant, computer-based control system, located at Susitna Hydroelectric Control Center will be provided;

- The Susitna in-plant control system will directly control all hydro generating units and the switching stations at Watana and Devil Canyon. EMS will determine generation participation requirements on the unit level, but the units will be pulsed by the in-plant system. The supervisory control actions for Watana and Devil Canyon generating stations will be initiated at EMS level, but the control functions will be implemented by the in-plant control system;

- The northern and southern computer-based systems will receive generation participation requirements from the EMS, but participation allocation and direct unit pulsing will be accomplished by these systems; and

- EMS will directly monitor and control the following 345 kV substations:

  . Ester;
  . Willow;
  . Knik Arm;
  . University; and
  . Others, as required.

(ii) Alternative II System Configuration

The Alternative II system configuration is also typical of current offerings of several EMS equipment manufacturers (see Figure 14.13, EMS Alternative II System Configuration). The configuration is based on the assumptions that:

- An in-plant, computer-based control system, located at Watana, will be provided to monitor generating units performance and control the units;

- All Watana and Devil Canyon generating units will be controlled (raise and lower) directly by EMS from system control center at Willow;

- All northern and southern area generating units will be directly controlled (raise and lower) by EMS, Willow Control Center; and

- The switching stations at Watana and Devil Canyon and the other four 345 kV substations will be directly monitored and controlled by the EMS Control Center.

(e) Communication Requirements

Effective operation of EMS is very dependent on transfer of data and immediate response of supervisory functions such as control and telemetering. Various communication systems to determine the most reliable and cost-effective communication media were evaluated.

(i) Power Line Carrier

This system is dependent on the state of the transmission line and, therefore, will not be available when the line is down.

(ii) Telephone

Telephone companies provide data transmission services but the service is very erratic and unreliable for EMS applications.

(iii) Microwave

A microwave system is the most reliable and cost-effective solution for the EMS communications. It is highly desirable to install a looped system for power system operation.

Microwave systems are line-of-sight propagation and have an average standard transmission path of approximately 35 to 40 miles in an area of flat terrain. The cost was estimated for approximately 17 towers and repeater stations and without having the benefit of a detail communication analysis. A microwave system is recommended for this application.

(f) System Software Requirements

The EMS should be provided will all the software required to satisfy the functional requirements described in paragraph (c) and all the software functions described below.

The software should be the general-purpose operating system, developed and tested by a major computer supplier and verified through many installations in real-time applications. It should provide a reliable, high-performance environment for the concurrent execution of multiuser, time-sharing, batch, and time-critical applications. This software will consist of the following major components:

- Executive services;
- System failover and system restart;
- Diagnostic programs;
- Programming services;
- Special data base, CRT display, and log/generation compilers;
- Engineering support; and
- Special I/O handlers.

Fortran compatibility of the software is essential, as most of the power application programs will be written in a high-level language.

(g) Control Center Facility

The facility will be the nerve center of the APA power system operations of 345 kV transmission network and the electric power generation. All decisions concerning the operation and maintenance of the power system will be implemented through this complex. The importance of this facility dictates that its location be selected with a great deal of care.

(i) Location of Site

The control center must be located on a site that provides high security against disruption of power system operations by human intervention or by acts of God. Acts of human intervention that must be considered are civil disturbances and terrorist activities. Natural disturbances that could occur are floods, fires, and landslides.

Several additional factors that have a bearing on the suitability of a site are:

- Land availability;
- Housing availability;
- Transportation accessibility;
- Educational facility availability;
- Climatic conditions;
- Power availability; and
- Centralized location in the power system.

All of the above factors and the fact that a major switchyard is already located in the area make it appropriate to recommend Willow as the location for the EMS center.

Willow has additional qualifications as a possible capital site. The Willow center could also be the headquarters for the maintenance staff for the transmission network between Susitna and Anchorage. The Willow site also has flat lands between it and Anchorage which also reinforces the recommendation to use microwave as the communication media.

(ii) Control Center Building

The EMS control center building can be located on the same site as the Willow switchyard. The construction of this building will require special facilities. This is all described in the "Energy Management System (EMS) - System Requirements" report.

Figure 14.4 provides a conceptual layout of the Willow Control Center. This layout is based on a one-level building having a total space of 14,537 ft^2.

(h) Staffing Requirements

The functional organization of the EMS Control Center must efficiently and comprehensively support all aspects of the operation and control of the Railbelt's power system. This also includes not only the day-to-day operations, but also the coordination of power transmission and generation and the ongoing training of personnel to improve efficiency and effectiveness.

(i) Power System Operations Staff

The following operating staff are recommended:

- One chief operator;
- Five senior operators;
- Nine load operators;
- One engineering technician; and
- One clerk.

The above organization can maintain a 24-hour operation for 365 days a year.

(ii) Computer Applications

The computer applications section should be managed by a supervisor of software applications. Reporting to this supervisor should be at least three additional software engineers charged with the duties of maintaining SCADA, Generation Control, and System Security software programs.

(iii) Power Coordination

The power coordination group will be responsible for evaluating unit commitment runs, preparing interchange schedules, and performing after-the-fact power accounting, etc. This group will include one supervisor, one power production specialist, one budget specialist, two power system engineers/analysts, two statisticians, and one power scheduler.

(iv) EMS Maintenance Group

The EMS maintenance group will be responsible for maintaining the EMS hardware and software. As a minimum, this group should include:

- One system hardware engineer;
- Two system software engineers;
- Two hardware technicians;
- Two RTU maintenance technicians; and
- One communication maintenance technician.

(j) Budgetary Cost Estimates

This paragraph provides overall budgetary cost estimates for the development procurement, system testing, and installation of EMS Alternatives I and II. Costs for the EMS Control Center and Microwave System are also provided. These costs are representative of what Energy and Control Consultants estimate as the middle price bids of such a project.

The cost estimates for these configurations, microwave system, and EMS Control Center are given in January, 1982 dollars for a fixed-price contract that includes milestone payments. Table 14.7 shows comparative cost estimates.

(k) Recommendations

Alternative I, shown in Figure 14.5, is recommended for the Railbelt Energy Management System as the most cost-effective and desirable system approach. Unlike Alternative II, Alternative I system approach allows generation control of the southern (Anchorage) and northern (Fairbanks) areas to remain under their respective utilities. Alternative I also encourages the formation of regional control centers for each area. Presently, this is the trend in power system control to decentralize in large geographical areas.

Alternative I is also marginally less costly than Alternative II.

Microwave is recommended as a communicating medium. Once provided, this system will perform the following additional functions:

- Provide a transmission media for protective line relaying; and
- Provide reliable voice communications between the various stations. This is very important in power system operations.

It is recommended that the EMS Control Center be located at Willow within the Willow Switching Station compound. This location has many advantages and is centrally located in the southern Railbelt power system. It would also be reasonable to designate this location as a maintenance center for the transmission system. This area has lots of land for expansion. There also appear to be some plans to provide a highway crossing at Knik Arm. If these plans materialize, Willow would only be one hour away by highway from Anchorage.

LIST OF REFERENCES

(1) Susitna Hydroelectric Project Planning Memorandum - Subtask 8.02 Preliminary Transmission System Analysis Acres, 1981

(2) Upper Susitna River Hydroelectric Studies Report on Transmission System U.S. Department of Interior Corps of Engineers, 1975

(3) Anchorage-Fairbanks Transmission Intertie Economic Feasibility Study Report IECo, 1979

(4) Terrain Analysis of the North and South Intertie Power Transmission Corridors, Prepared for Acres American, Inc., and the Alaska Power Authority by R&M Consultants, Inc., Anchorage, Alaska.

TABLE 14.1: POWER TRANSFER REQUIREMENTS (MW)

	INSTALLED CAPACITY			TRANSFER REQUIREMENT	
Year	Watana	Devil Canyon	Total Susitna	Susitna to Anchorage	Susitna to Fairbanks
1993	680	--	680	578	170
1997	1020	--	1020	867	255
2002	1020	600	1620	1377	405

TABLE 14.2: SUMMARY OF LIFE CYCLE COSTS

TRANSMISSION ALTERNATIVE	1	2	3	4	5
Transmission Lines			1981 $ x 10^6		
Capital	$156.70	$159.51	$133.96	$140.94	$159.27
Land Acquisition	18.73	20.79	18.07	20.13	18.65
Capitalized Annual Charges	127.34	130.14	107.43	112.83	126.91
Capitalized Line Losses	53.07	54.50	64.51	65.82	42.82
Total Transmission Line Cost	$355.84	$364.94	$323.97	$339.72	$347.65
Switching Stations					
Capital	$114.09	$106.40	$128.32	$120.64	$154.75
Capitalized Annual Charges	121.02	113.30	135.94	128.22	165.02
Total Switching Station Cost	235.11	219.70	264.26	248.86	319.77
Susitna Life Cycle Cost	$590.95	$584.64	$588.23	$588.58	$667.42

TABLE 14.3: TRANSMISSION SYSTEM CHARACTERISTICS

Line Section	Length (mi)	Number of Circuits	Voltage (kV)	Number & Size of Conductors (kcmil)
Watana to Devil Canyon	27	2	345	2 by 954
Devil Canyon to Fairbanks	189	2	345	2 by 795
Devil Canyon to Willow	90	3	345	2 by 954
Willow to Knik Arm	38	3	345	2 by 954
Knik Arm Crossing*	4	3	345	———
Knik Arm to University Substation	18	2	345	2 by 1351

*Submarine Cable

TABLE 14.4: TECHNICAL, ECONOMIC, AND ENVIRONMENTAL CRITERIA USED IN CORRIDOR SELECTION

Type	Criteria	Selection
1. Technical		
- Primary	General Location	Connect with Intertie near Gold Creek, Willow, and Healy. Connect Healy to Fairbanks. Connect Willow to Anchorage.
	Elevation	Avoid mountainous areas.
	Relief	Select gentle relief.
	Access	Locate in proximity to existing transportation corridors to facilitate maintenance and repairs.
- Secondary	River Crossings	Minimize wide crossings.
2. Economical		
- Primary	Elevation	Avoid mountainous areas.
	Access	Locate in proximity to existing transportation corridors to reduce construction costs.
- Secondary	River Crossings	Minimize wide crossings.
	Timbered Areas	Minimize such areas to reduce clearing costs.
	Wetlands	Minimize crossings which require special designs.
3. Environmental		
- Primary	Development	Avoid existing or proposed developed areas.
	Existing Transmission Right-of-Way	Parallel.
	Land Status	Avoid private lands, wildlife refuges, parks.
	Topography	Select gentle relief.
- Secondary	Vegetation	Avoid heavily timbered areas.

TABLE 14.5: TECHNICAL, ECONOMIC AND ENVIRONMENTAL CRITERIA USED IN CORRIDOR SCREENING

Technical

Primary

- Topography
- Climate and Elevation
- Soils
- Length

Secondary

- Vegetation and Clearing
- Highway and River Crossings

Economic

Primary

- Length
- Presence of Right-of-Way
- Presence of Access Roads

Secondary

- Topography
- Stream Crossings
- Highway and Railroad Crossings

Environmental

Primary

- Aesthetic and Visual
- Land Use
- Presence of Existing Right-of-Way
- Existing and Proposed Development

Secondary

- Length
- Topography
- Soils
- Cultural Reservoir
- Vegetation
- Fishery Resources
- Wildlife Resources

TABLE 14.6: SUMMARY OF SCREENING RESULTS

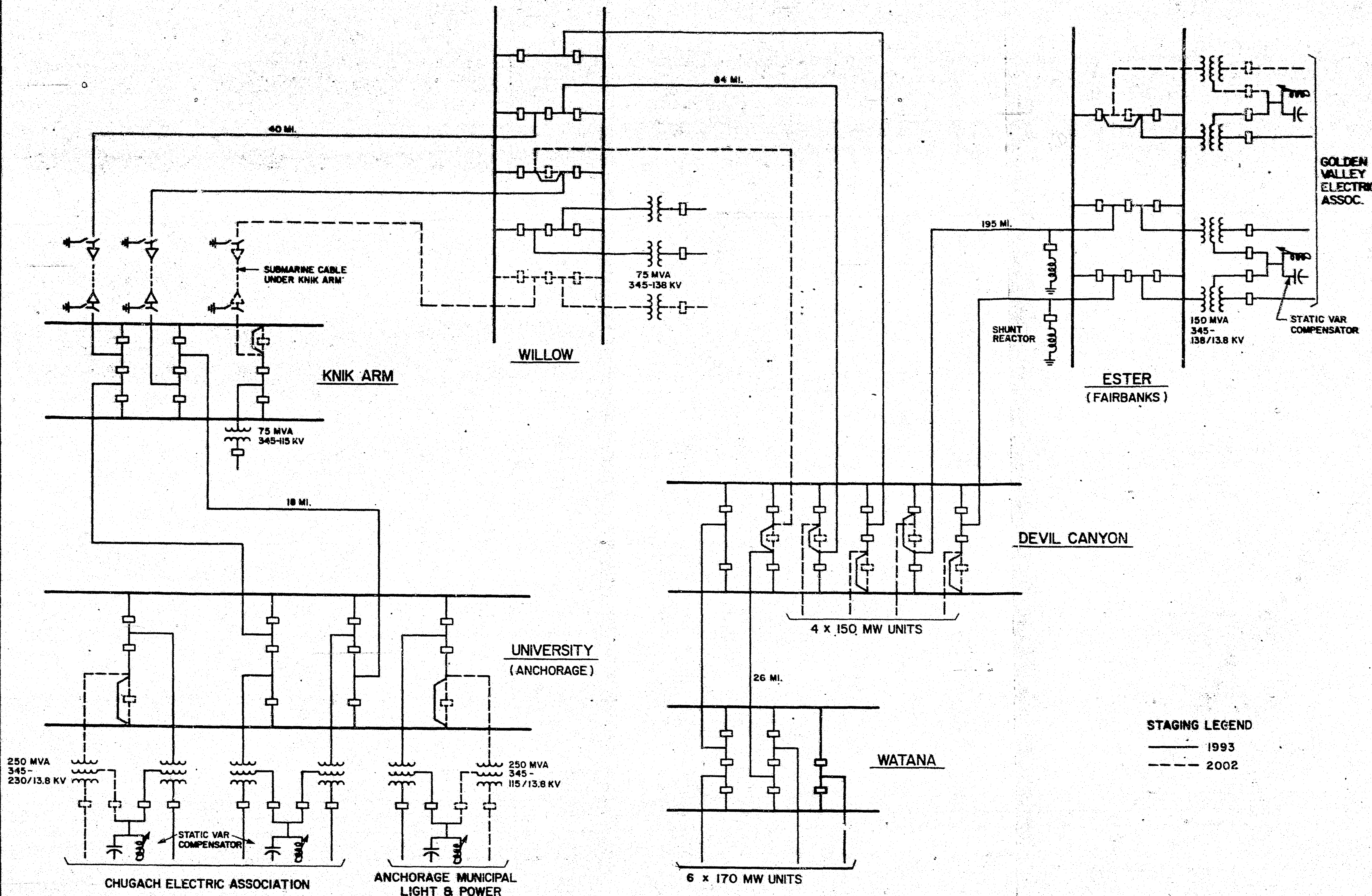
Corridor	Env.	Econ.	Tech.	Summary
	RATINGS			
- Southern Study Area				
(1) ABC'	C	C	C	C
*(2) ADFC	A	A	A	A
(3) AEFC	F	C	A	F
- Cental Study Area				
*(1) ABCD	A	A	A	A
(2) ABECD	F	C	C	F
(3) AJCF	C	C	C	C
(4) ABCJHI	F	F	F	F
(5) ABECJHI	F	F	F	F
(6) CBAHI	F	C	F	F
(7) CEBAHI	F	F	C	F
(8) CBAG	F	F	C	F
(9) CEBAG	F	F	C	F
(10) CJAG	F	F	C	F
(11) CJAHI	F	C	C	F
(12) JACJHI	F	F	C	F
(13) ABCF	A	C	A	C
(14) AJCD	C	A	A	C
(15) ABECF	F	C	C	F
- Northern Study Area				
*(1) ABC	A	A	A	A
(2) ABDC	C	A	C	C
(3) AEDC	F	C	F	F
(4) AEF	F	C	F	F

A = recommended
C = acceptable but not preferred
F = unacceptable

*Indicates selected corridor.

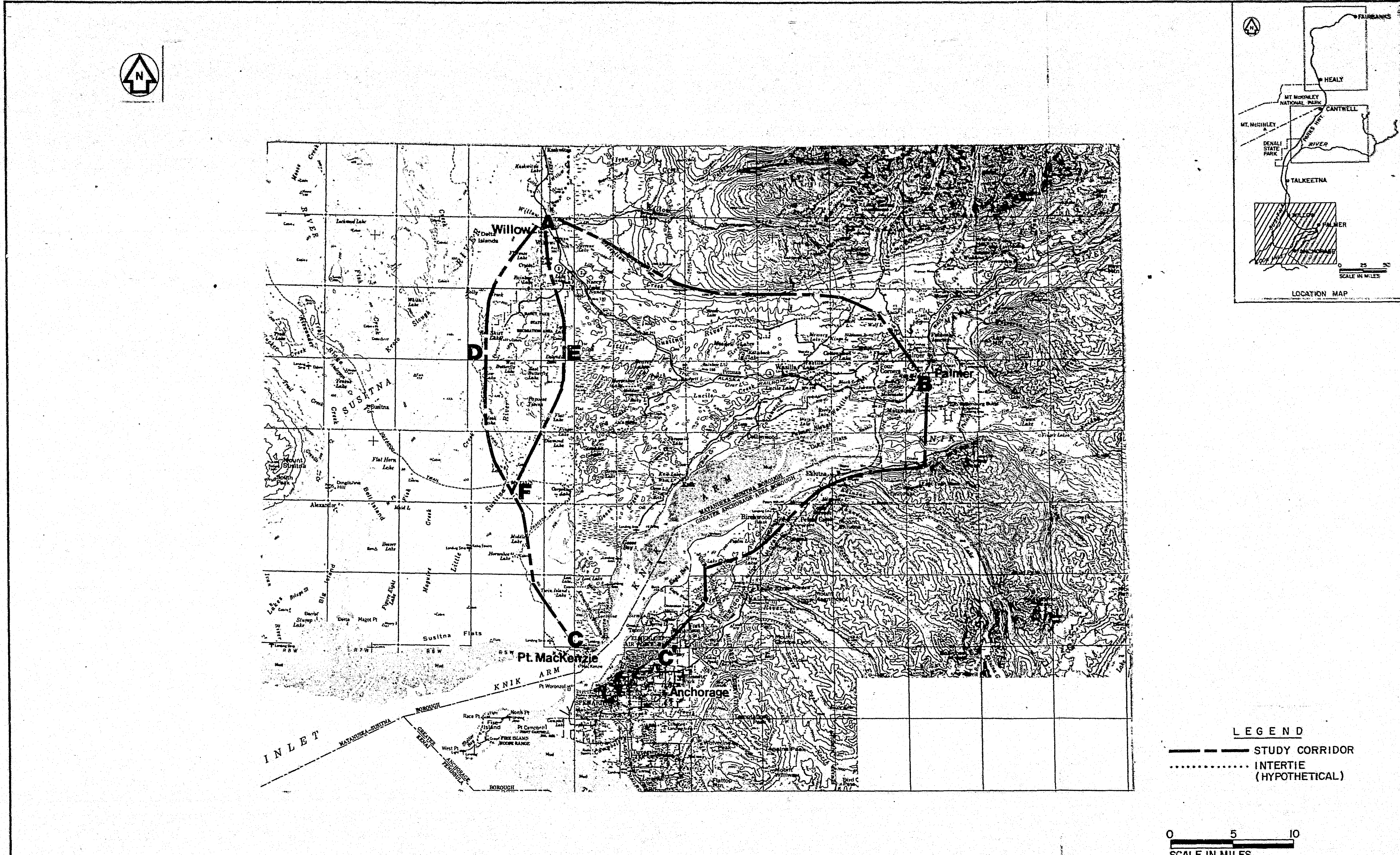
TABLE 14.7: EMS ALTERNATIVES I AND II COMPARATIVE COST ESTIMATES

	Alternative I	Alternative II
EMS Project		
Hardware	$ 2,942,000	$ 3,072,000
Software	3,956,000	4,200,000
Auxiliary	1,210,000	1,350,000
Internal (APA costs)	3,416,000	3,606,000
	$ 11,524,000	$ 12,228,000
Susitna In-Plant Control System		
Hardware	$ 1,131,000	$ 1,094,000
Software	1,200,000	1,200,000
Auxiliary	750,000	700,000
Internal (APA costs)	1,770,000	1,875,000
	$ 4,851,000	$ 4,869,000
Microwave System	$ 4,920,000	$ 5,100,000
EMS Control Center Building	$ 3,853,140	$ 3,853,140
TOTAL	$ 25,148,140	$ 26,050,140



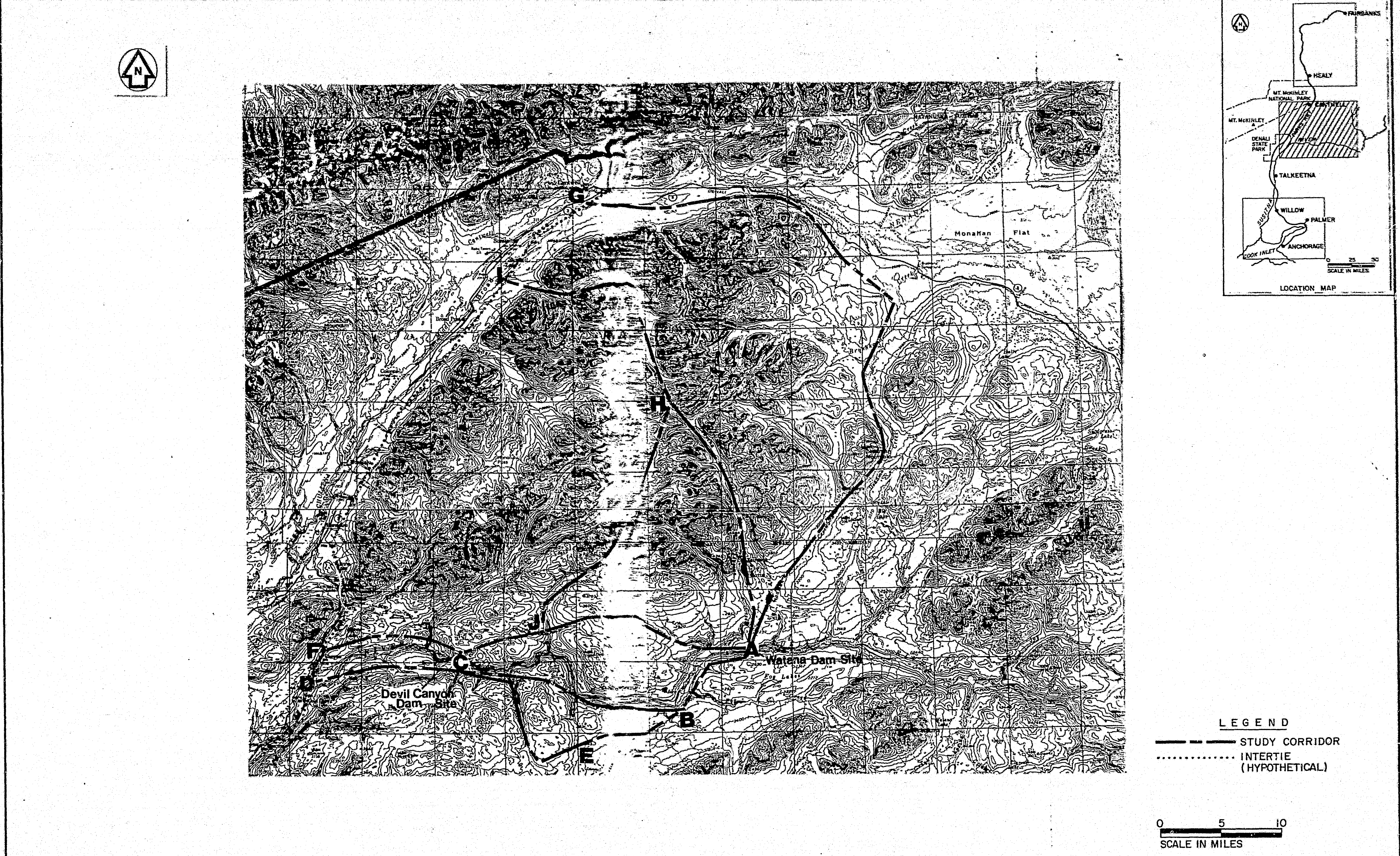
RAILBELT 345 KV TRANSMISSION SYSTEM SINGLE LINE DIAGRAM

FIGURE 14.1



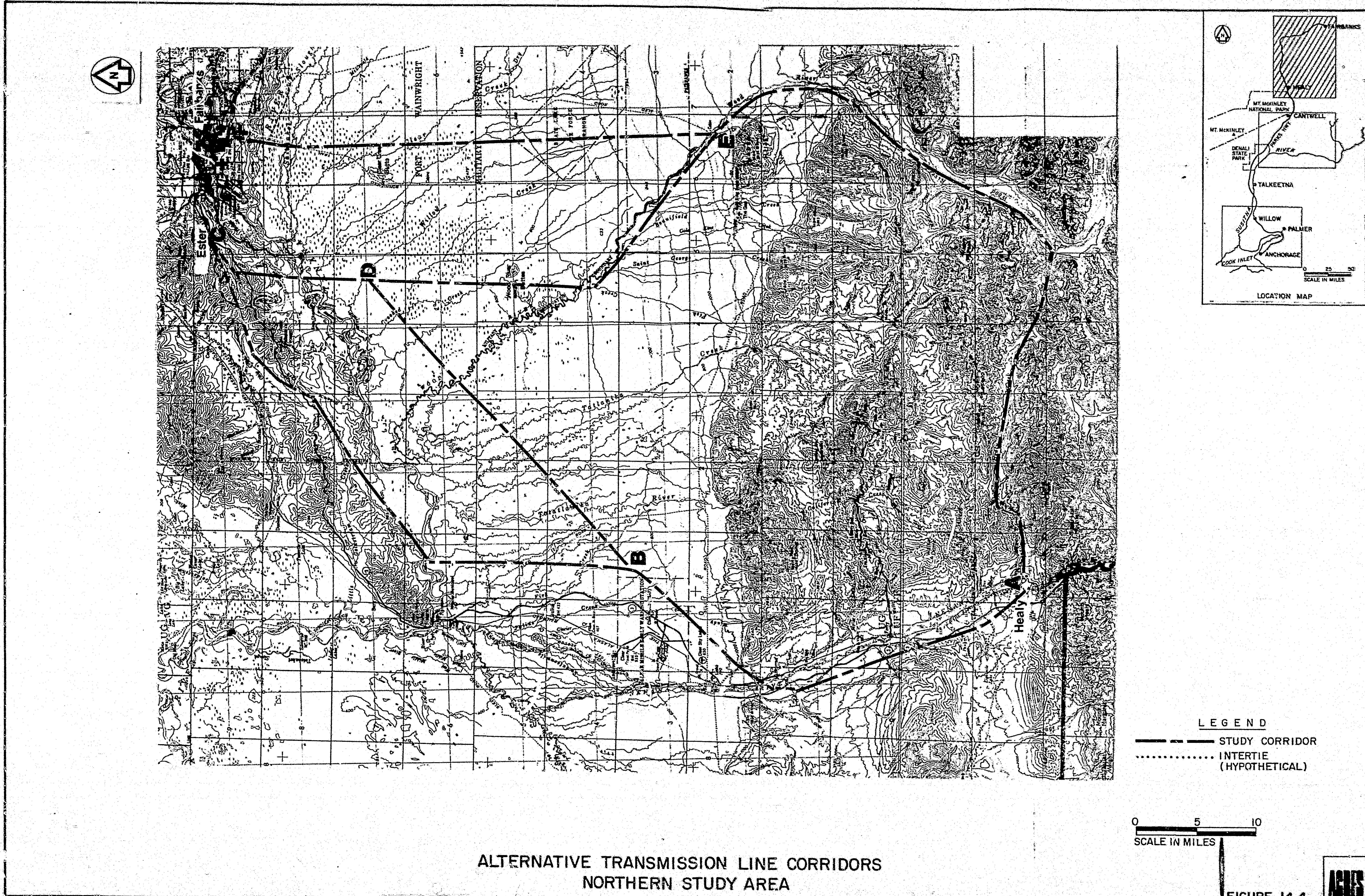
ALTERNATIVE TRANSMISSION LINE CORRIDORS
SOUTHERN STUDY AREA

FIGURE 14.2



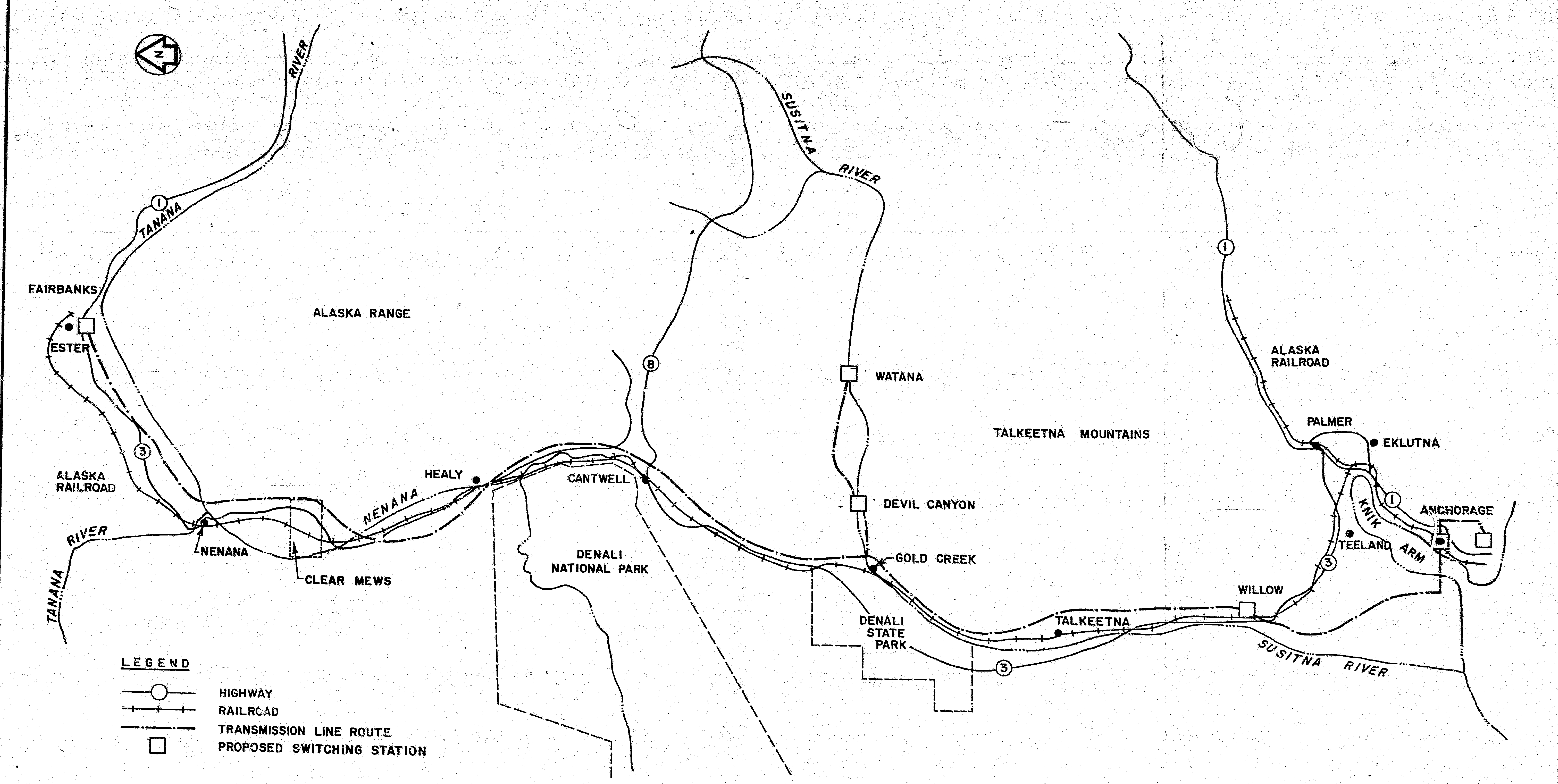
ALTERNATIVE TRANSMISSION LINE CORRIDORS
CENTRAL STUDY AREA

FIGURE 14.5



ALTERNATIVE TRANSMISSION LINE CORRIDORS
NORTHERN STUDY AREA

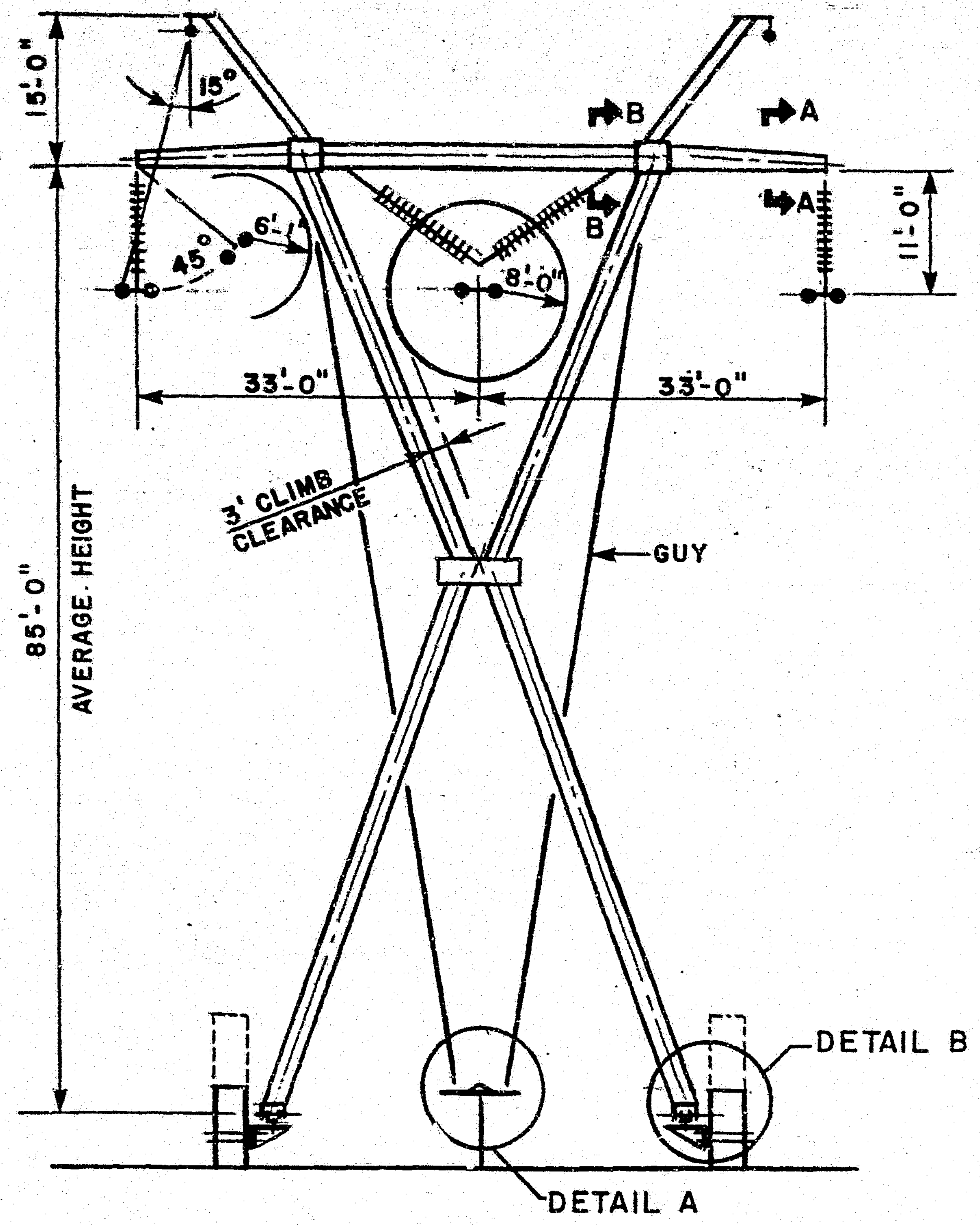
FIGURE 14.4



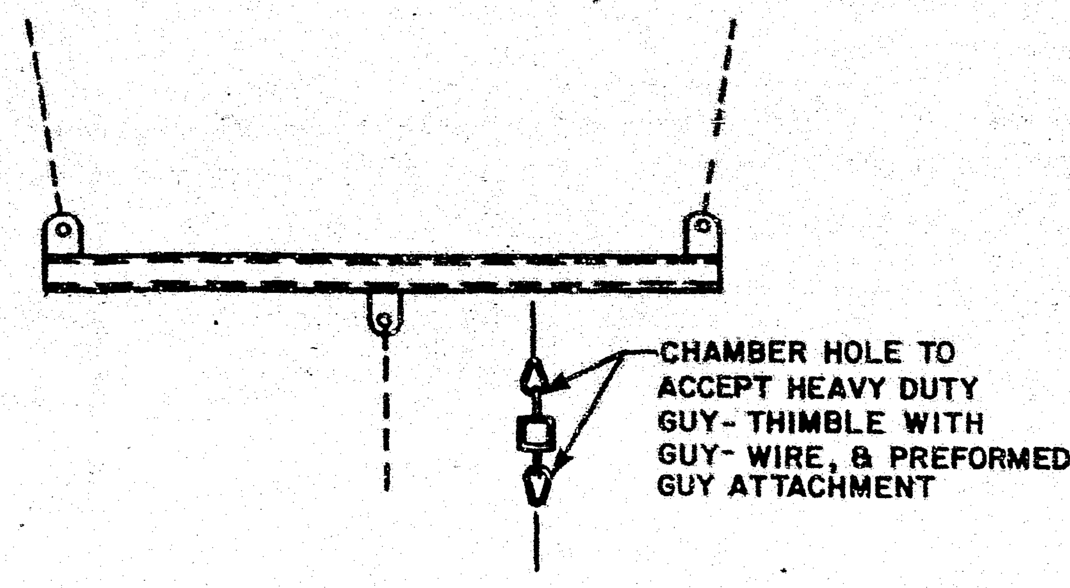
THE PROPOSED INTERTIE FROM WILLOW TO HEALY WILL BE CONSTRUCTED FOR 345 KV CAPABILITY AND INITIALLY OPERATED AT 138 KV.
IF SUSITNA IS PROVED FEASIBLE, THE FULL 345 KV CAPACITY WILL BE UTILIZED.

ANCHORAGE TO FAIRBANKS
PROPOSED TRANSMISSION LINE ROUTE

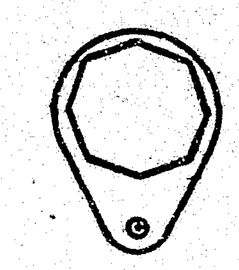
FIGURE 14.5



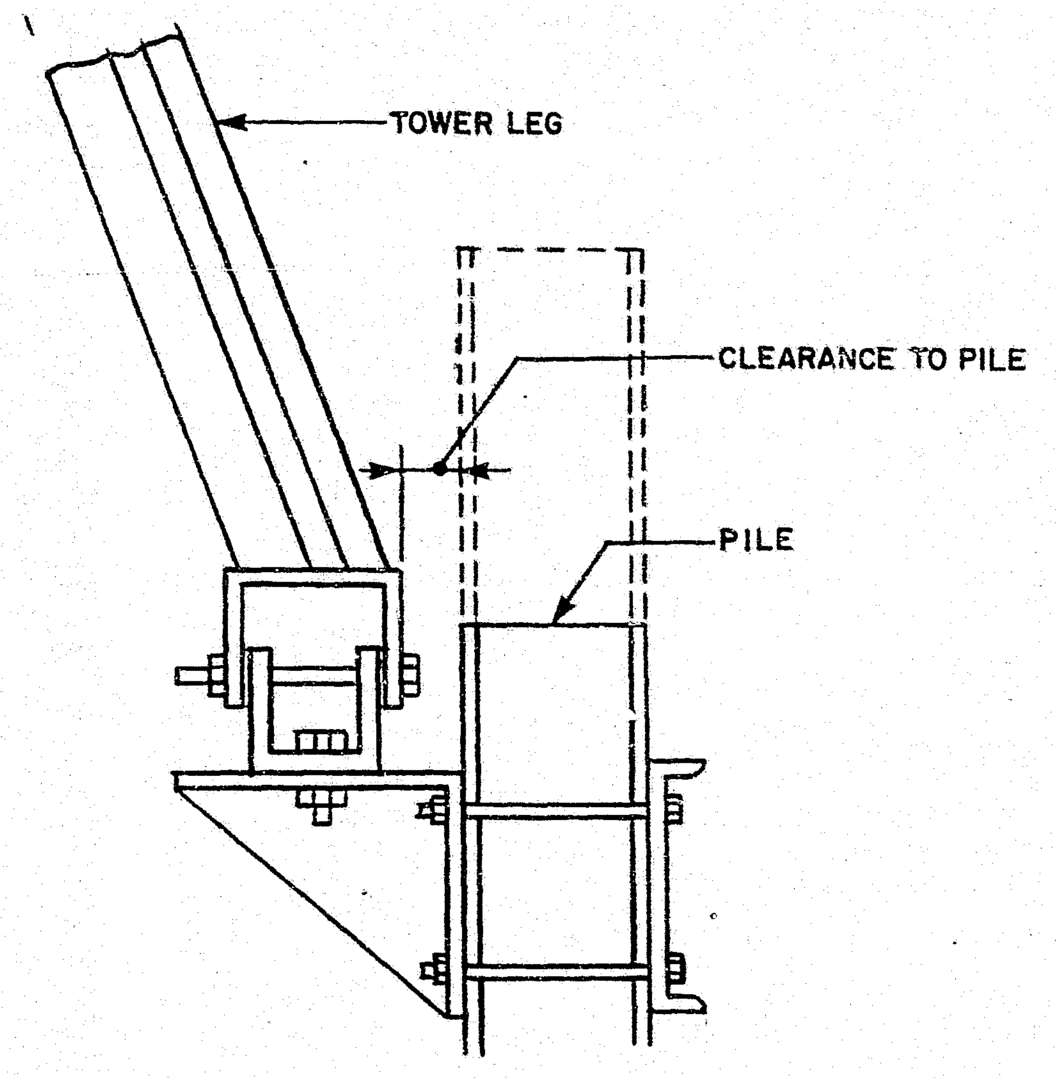
RECOMMENDED 345 KV TANGENT TOWER



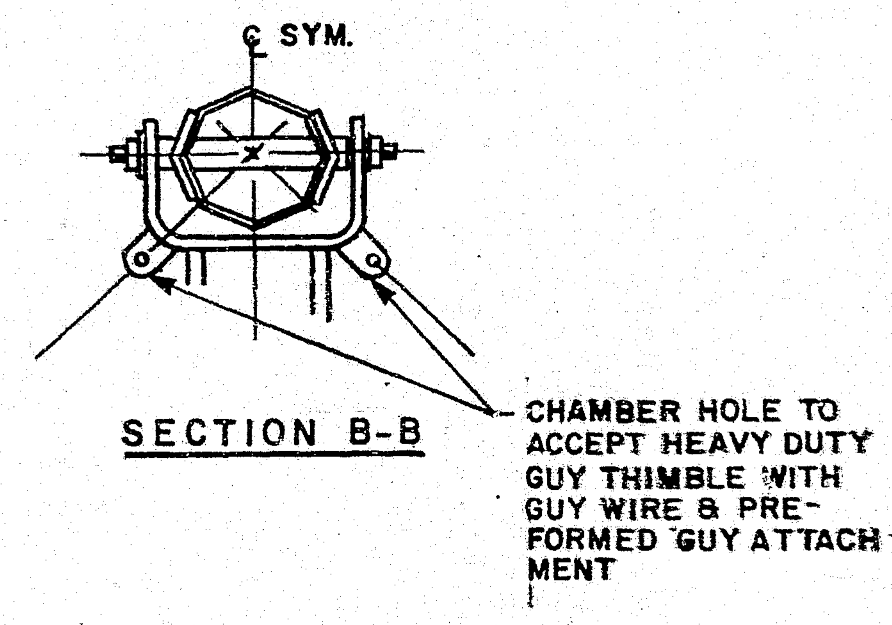
DETAIL A



SECTION A-A



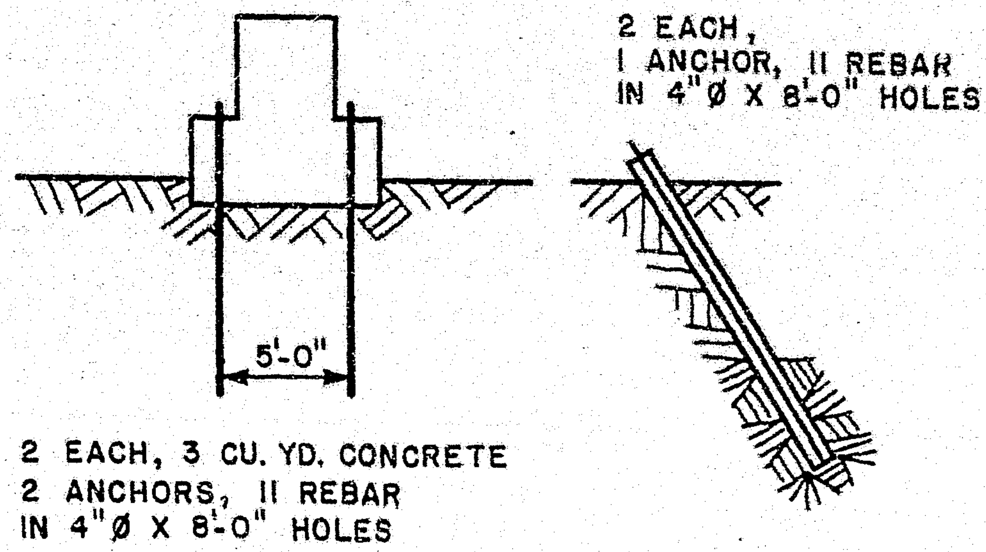
DETAIL B



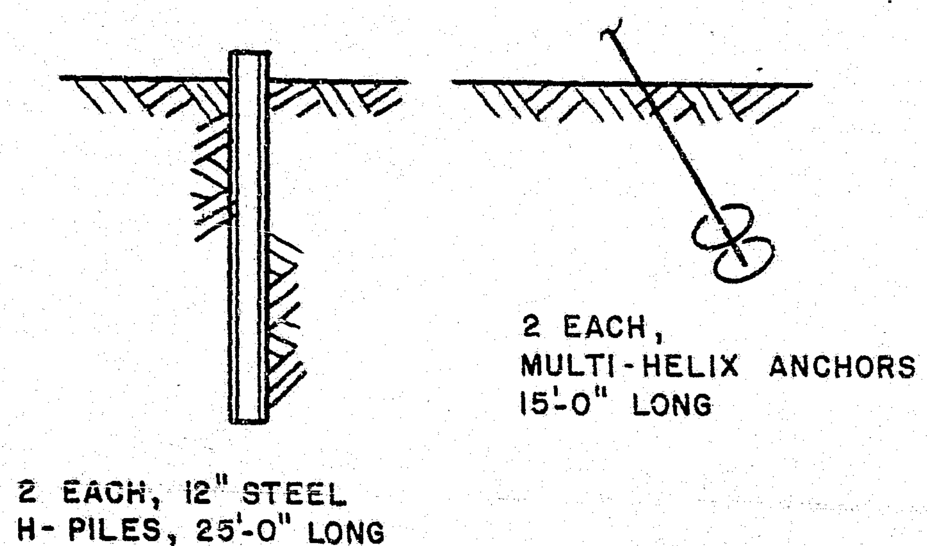
SECTION B-B

X - FRAME GUYED STEEL TOWER

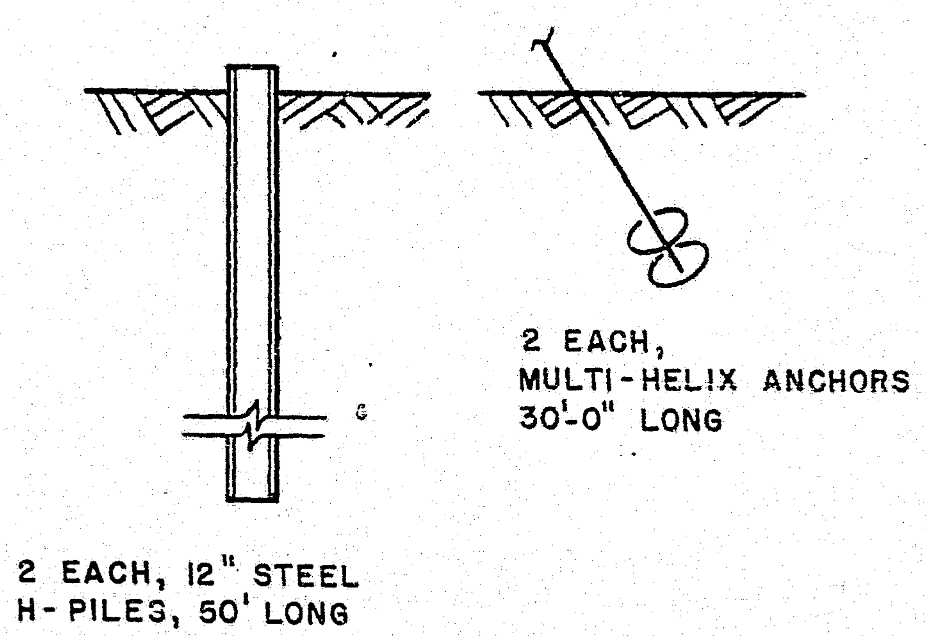
FIGURE 14.6



ROCK FOUNDATION



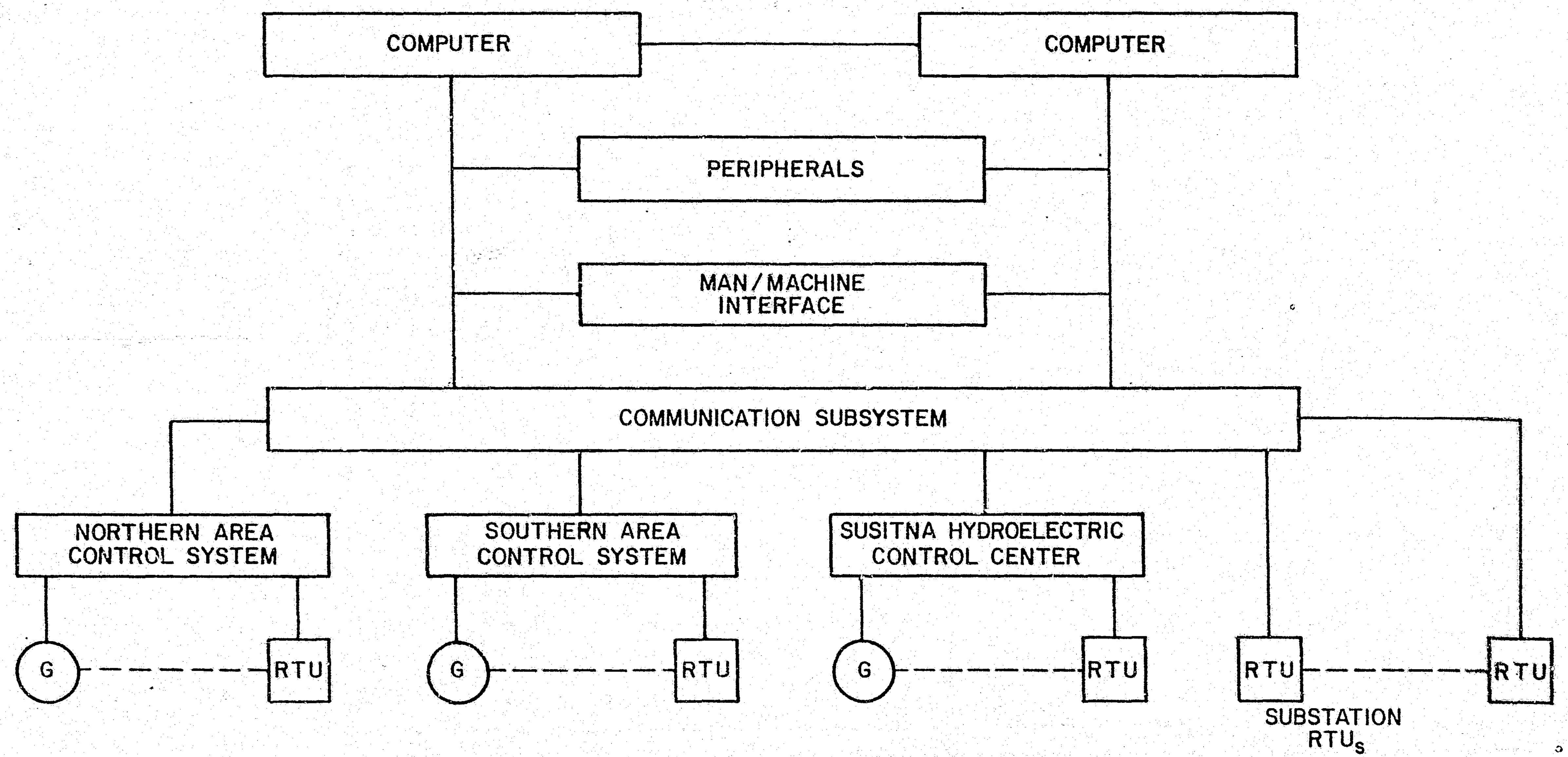
STANDARD FOUNDATION



WET LAND FOUNDATION

TRANSMISSION TOWER FOUNDATION CONCEPTS

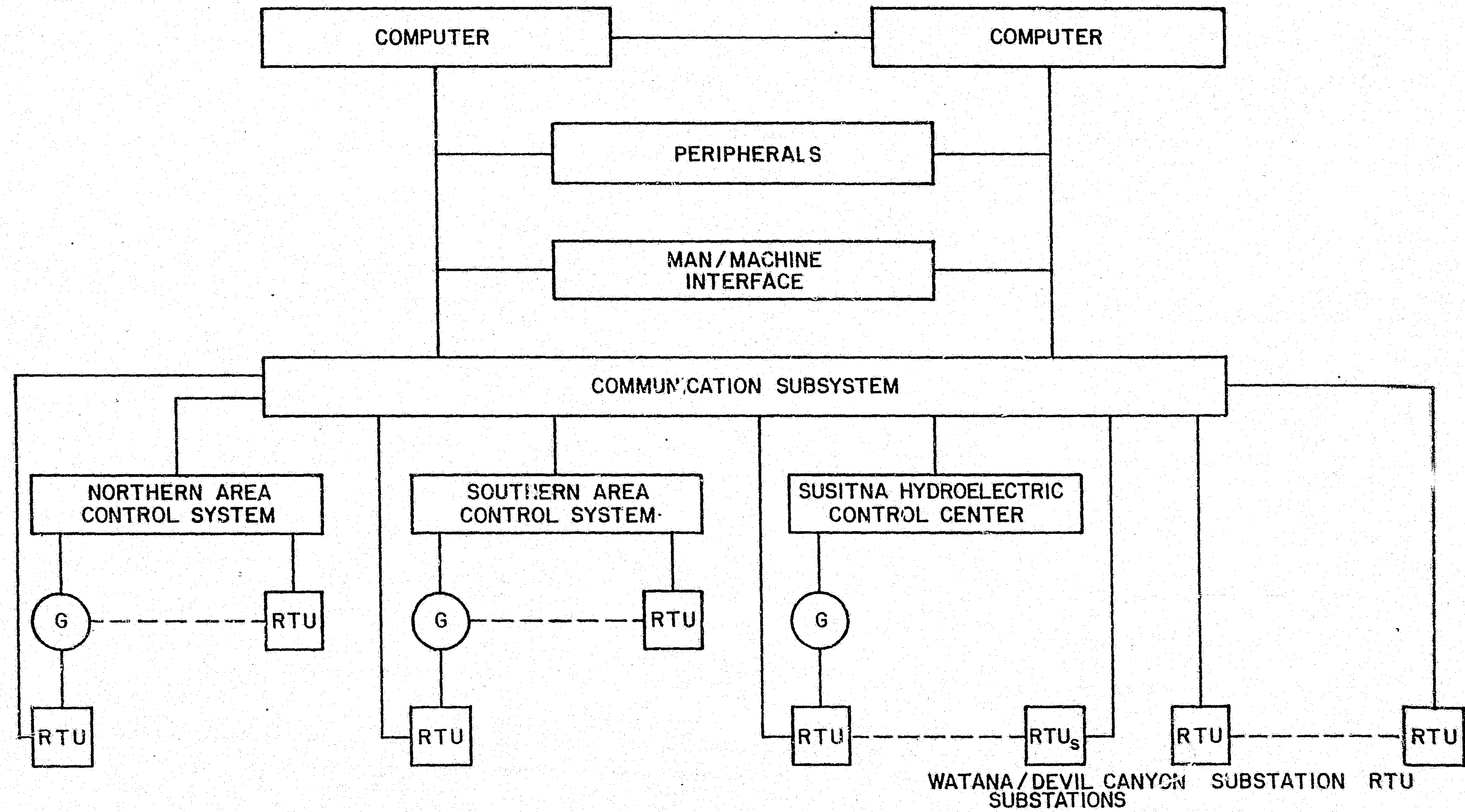
FIGURE 14.7



ENERGY MANAGEMENT SYSTEM, ALTERNATIVE I, SYSTEM CONFIGURATION

FIGURE 14.12

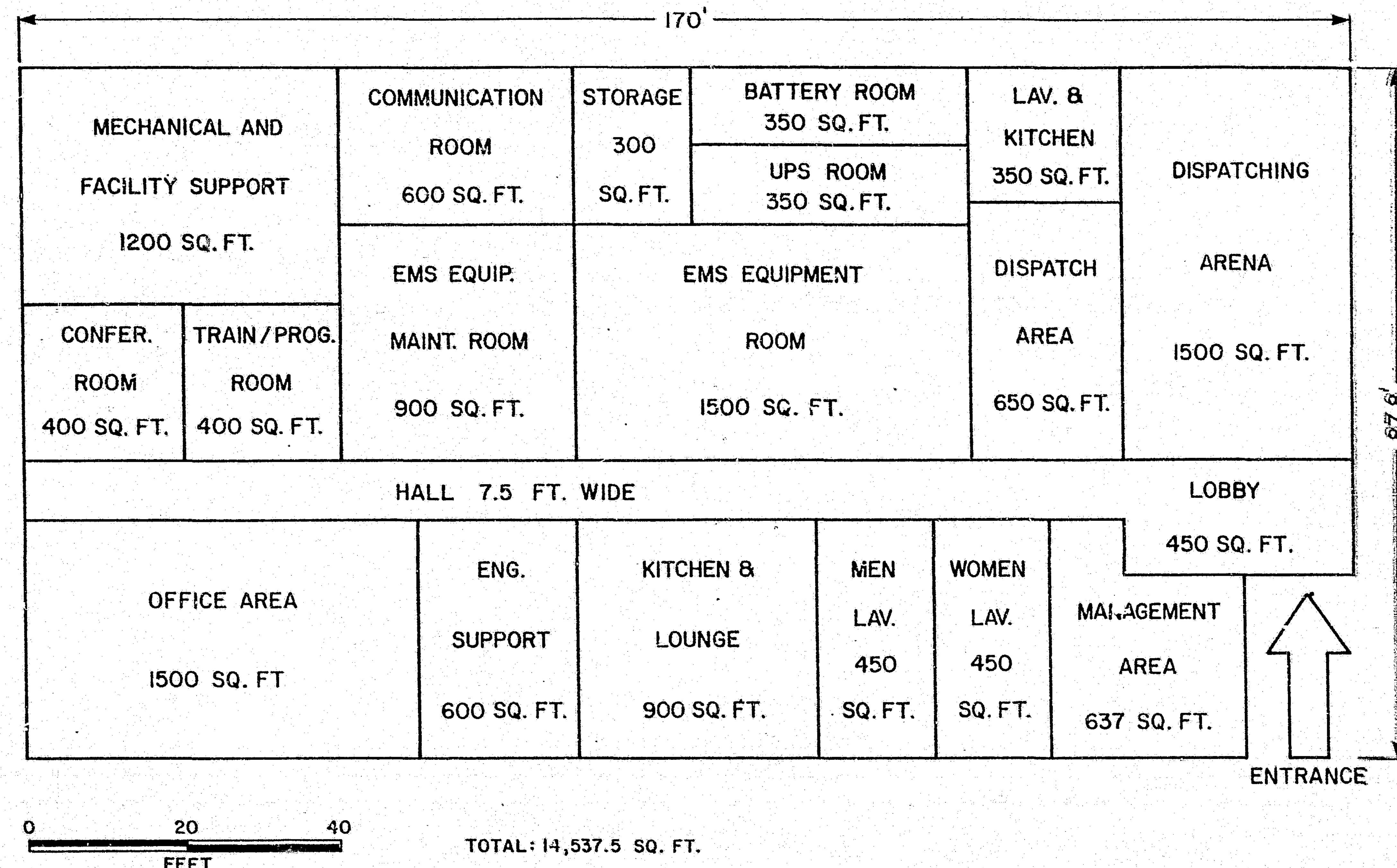
ACRES



ENERGY MANAGEMENT SYSTEM, ALTERNATIVE II, SYSTEM CONFIGURATION

FIGURE 14.13

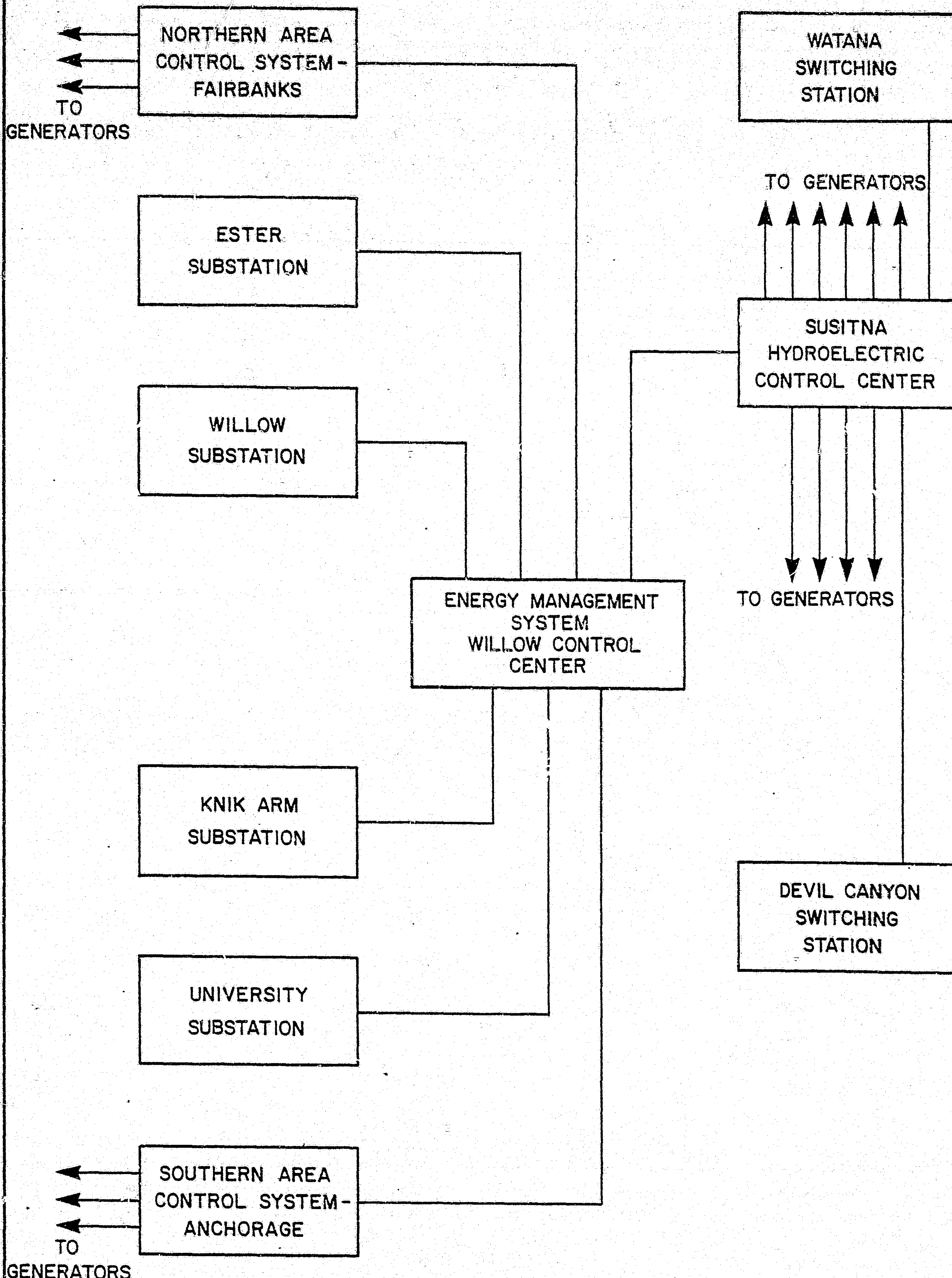
ACRES



WILLOW SYSTEM CONTROL CENTER,
FUNCTIONAL LAYOUT

FIGURE 14.14

ACRES



ENERGY MANAGEMENT SYSTEM, ALTERNATIVE I,
CONFIGURATION BLOCK DIAGRAM

FIGURE 14.15

ACRES

15 - PROJECT OPERATION

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 development in Sections 6, 8, and 14 of the Susitna project, size and schedule of on-line dates, and the associated transmission line requirements was necessarily based on the assumption that a single entity would eventually be set up to optimize and control the dispatch and distribution of electric power from all Railbelt sources. It is not the purpose of this report to discuss how this entity should be structured or come about. However, it is important to note that the Susitna project will be the single most significant power source in the System. Careful consideration is therefore essential of the dispatch and distribution of power from all sources by the most economical and reliable means. The general principles of reliability of plant and system operation, plant operation and reservoir regulation, stationary and spinning reserve requirements and maintenance programming are also discussed. Estimates of dependable capacity and annual energy production for both Watana and Devil Canyon are presented. Operating and Maintenance facilities and procedures are described and the proposed performance monitoring system for the two projects is also outlined.

15.1 - Plant and System Operation Requirements

The two plants comprising the Susitna Project will represent about 75 percent of the system capacity, having an installed capacity of 1620 MW in a total system installed capacity of 2100 MW in the year 2010. In view of its large capacity and the extent of its influence on the operational characteristics of the power system, it is appropriate that the Susitna project operation should be discussed within the framework of general power system operation considerations of economy and security.

Planning studies discussed in Section 8 were primarily concerned with selection of plant installed capacity such that an optimum installation could be provided to meet projected generation requirements over the life of the project, which may be considered as 50 years or more.

The main function of system planning and operation control to be discussed in this section is concerned with the allocation of generating plant in the system on a short-term operation basis so that the total system load demand is met by the available generation at minimum cost consistent with the security of supply.

The general 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 operational case the actual state of the system dictates system security requirements overriding economic considerations in load dispatching. An important factor arising from economic and security considerations in the system planning and operation is the provision of reserve capacity, both as stationary reserve as well as spinning reserve.

The basis of system operation is the demand to be met at any moment and is the aggregate of all consumers' demands in the interconnected Railbelt System. Figure 15.1 shows the daily variation in demand during typical winter and summer weekdays and the seasonal variation in monthly peak demands for estimated loads in a typical year (the year 2000).

15.2 - General Power Plant and System Railbelt Criteria

The power plants and electrical system are planned and constructed in such a manner that they can be operated so that the more probable contingencies can be sustained without loss of load. Less probable contingencies are also examined and the consequences to the system are determined and evaluated. The more probable contingencies are usually defined as those that occur once in 5 or 10 years. The less probable contingencies have a probability of occurrence of once in 50 or 100 years. Automatic load relief (and when necessary in extreme cases, load shedding) is provided to minimize the probability of total shutdown of area load which becomes isolated by multiple contingencies. The following are basic reliability standards and criteria generally adopted in the industry for power systems. Further details are described in Section 8 for generation planning, in Sections 12 and 13 for the power plants, and in Section 14 for the transmission system.

(a) Installed Generating Capacity

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

The factors affecting the calculation of probability include the characteristics of the loads, the probability of error in load forecast, the scheduled maintenance requirements for generating units, the forced outage rates of generating units, limited energy capacity of plants, effects of interconnections, and transmission transfer capabilities. The calculation of LOLP is done in the generation planning studies described in Section 8.

(b) Transmission System Capability

The high-voltage transmission system in general and that associated with the project in particular, 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.

- 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 are 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.

It is impossible to anticipate or test the system for all contingencies that can occur in present or future configurations. Typical examples of the less probable contingencies are:

- Sudden loss of the entire generating capability of any power plant for any reasons.

- Sudden loss of all transmission lines on a single right of way.

- Sudden dropping of a very large load at a major load center.

The above stated general principles of reliability and security have been generally applied in the design of the project and system and are described and referred to in greater detail in the various pertinent sections of this report. These principles constitute the basis of project operation planning and criteria described in this section and elsewhere in the report.

(c) Summary

Operational reliability criteria thus fall into four main categories:

(i) Loss-of-load probability (LOLP) of 0.1 or one day in ten years, is maintained for the recommended plan of Susitna project and system operation through the year 2010 (Section 6, Generation Planning).

(ii) The single and double contingency requirements are maintained for any of the more probable outages in the plant or transmission system.

(iii) System stability and voltage regulation are assured from the electrical studies (Section 14, Transmission Facilities). Detailed studies for load frequency control have not been done, but it is expected that the stipulated criteria will be met with the more than adequate spinning reserve capability with six units at Watana and four units at Devil Canyon.

(iv) The loss of all Susitna transmission lines on a single right-of-way has a low level of probability as described in Section 18 under Risk Analysis. In the event of the loss of all lines, the hydroplants 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 plants are superior to thermal plants because of their inherent black start capability for restoration of supply to a large system.

15.3 - Economic Operation of Units

The Central Dispatch Control engineer has the responsibility of deciding which generating units should be run at any particular time. Decisions are made on the basis of a number of different pieces of 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. Area demand forecasts up to 8 hours ahead of unit loading are based on regional short-range weather forecasts for an estimate of heating and lighting demands plus light or heavy industry loads. Short-term forecasting up to 1 or 2 hours ahead is more difficult and remains the key factor to the secure and economic operation of the system.

Based on the demand, basic power transfers between areas and an allowance for reserve, the tentative amount of generating plant is determined, 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 Susitna case, the hydro unit will take up a much greater part of base load operation than in a thermal dominated power system. The 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.

No significant load following can be done by the Devil Canyon units due to environmental constraints as described in Section 15.6.

(c) Operating Limits of Units

Ideally, the plant having the lowest fuel costs should be allocated load for as long as possible, and the most expensive plant required to meet the peak demands for as short a time as possible. In practice, it is not possible to meet this ideal situation due to security requirements for the system and the characteristics of the generation plant. There are strict constraints on the minimum load and the loading rates of machines and to dispatch load to these machines requires a system wide 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 rough zone of turbine operation (typically from speed-no-load to 50 percent 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.

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

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

(iii) Gas Turbines

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

- Requires 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. Monthly, yearly, and 5-year maintenance schedules are prepared. In a large interconnected system with minimum reserves, optimum maintenance programming uses heuristic methods. The program planning takes into consideration the availability of trained repair and/or maintenance personnel. Further details of Watana and Devil Canyon power plant maintenance programs are given in subsection 15.8.

15.4 - Unit Operation Security Criteria

During the operational load dispatching conditions of the power system, the security criteria often override the economic considerations of merit-order scheduling of the various units in the system. It is impossible to anticipate all the probable contingencies of operation, hence an operational approach implies the use of continuous on-line data updating the state of the system for the information of the operator. Projected electrical power system analyses are then carried out at frequent intervals to estimate the security of operation. Also important in consideration of operational security are system response, load-frequency control and spinning reserve capabilities.

(a) Power System Analyses

During the planning stages, system studies are carried out for all credible generation and network changes and probable contingencies. The transmission system studies (Section 14) were undertaken to check the more probable cases for load flow, short circuit and transient stability. The load flow studies determined the voltage levels and reactive power compensation for various plant loadings to meet the various load demands up to the year 2010. The transient stability study determined that the system was stable due to a transmission system fault resulting in the outage of a critical transmission line.

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 statutory 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 5/3 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. The factor 5/3 allows for the maximum dip in the frequency of the system. 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).

Upon sudden loss of generation, additional power may initially be derived from the inertia of the rotating machines (in the first few seconds) and then from the spinning reserve depending on its governor action in the next 10 to 20 seconds. The minimum system frequency should in the meantime be contained within the specified statutory limits till steady state operation is reached.

From preliminary studies regarding the plant response in the system, it appears that Watana is best suited for system frequency control and regulation. The Watana unit being the largest in the system could be put under sensitive governor control. Small generating stations and base-load thermal plant would be normally given steep governor drops so that they maintain their scheduled power output despite small changes in frequency.

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.

Detailed load-frequency and spinning reserve studies should be done in the design stage of the project.

(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 not to restrict the normal or necessary network transfer capabilities of the power system.

15.5 - 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.

The details of the Central Dispatch Control Center and of the Watana Area Control Center are given in Section 14. 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.

15.6 - Susitna Project Operation

A reservoir simulation model was used to evaluate the optimum method of operation of the Susitna reservoirs and power plants at Watana and Devil Canyon. 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. Details of the computer model are described in Appendix B.2. The following paragraphs summarize the main features of reservoir operation.

(a) Reservoir Operation

Gross storage volume of the Watana reservoir at its normal maximum operating level of 2185 feet is about 9.5 million ac/ft which is about 1.6 times the mean annual stream runoff (MAF) in the river at the dam site. Live storage of 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 reservoir simulation model uses estimated historical monthly streamflows at the damsites for 32 years of available records, reservoir characteristics, and power facility parameters as a basis of estimating the energy potentials of the developments. Hydrological, environmental, equipment and geotechnical constraints were incorporated in the simulation to take account of varied requirements.

(i) Hydrological Constraints

The 32-year records of hydrology simulates seasonal and over-the-year flow characteristics of the river and as outlined in Section 7.2., includes a series of very dry years and correspondingly lower energy potential in those years.

(ii) Environmental Constraints

A variety of environmental constraints were developed and evaluated during the course of the study and the following have been incorporated in the reservoir operations:

- To reduce fluctuations in downstream river flows and water levels, no significant daily load following will be attempted from the Devil Canyon power station;

- To maintain proposed fisheries mitigation efforts (further discussion of flows to minimize impacts on downstream fisheries is presented in Volume 2 of this report) a minimum flow of 5,000 cfs will be maintained at Gold Creek at all times during the reservoir operation.

- A minimum flow of 500 cfs will be maintained at all times in the river reach between the Devil Canyon dam and the Devil Canyon tailrace outlet.

- Both Watana and Devil Canyon reservoirs will be operated throughout spring and summer months to attenuate flood discharges to the extent possible. This will minimize potential damage to fisheries mitigation efforts due to high flood peak discharges.

(iii) Equipment Constraints

Generating equipment in each powerhouse will be assumed to function at not less than 50 percent of maximum output to avoid rough operation.

(iv) Geotechnical Constraints

The assumed ratio of reservoir drawdown and filling have been limited to ensure that no serious reservoir slope stability problems will occur. In addition, maximum drawdown limits have been determined which will produce the optimum combination of firm and average energy. The drawdown limits are discussed in more detail in Appendix B2.

(b) System Demand and Reservoir Operating Rules

Studies of reservoir operation were based on preliminary demand forecasts established for the initial development selection studies (Section 5) and subsequently revised to take account of the system load forecast developed in power alternative studies by Battelle (Section 5.7). System reliability criteria (Section 15.2) requires a guaranteed or firm energy to be available from the Watana and Devil Canyon developments. This energy is a function of hydrology of the river, reservoir storage and operating procedures. The reservoir simulation model uses a procedure to maximize the firm energy potential of the developments, consistent with the various constraints listed above.

An optimum reservoir operation requirement was thus established by an iterative process to minimize net system operating costs while maximizing firm and usable energy production during the earlier years of demand growth. Four alternative operating rules 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" or "avoidance flows". Cases B and C are intermediate levels of power operation and downstream impact. These essentially define monthly minimum reservoir levels that should be maintained to provide firm energy consistent with constraints outlined above. For feasibility report purposes, operation model "A" was adopted for project design and approximate fisheries mitigati on measures developed (See Volume II). Details of the computer simulation runs for energy potential and their impact on project economics may be found in Appendix B.2. Table 15.1 presents a summary of potential energy generation with different operating rules for Watana and Devil Canyon developments.

The proposed reservoir operating rule (Case A) is presented in Figure 15.1. This mode of operation represents target minimum levels to be strived for in the operation of the reservoir. The target Watana reservoir, level to be attained at the end of September each year is fixed at 2,190 feet. This level of 5 feet above the normal maximum operating level is designed to provide a higher level of winter energy production to meet the greater winter demand. This is consistent with river hydrology in that significant floods do not occur in the period from October to early May, and safety of structures is not sacrificed.

For this mode of operation, the average annual drawdown in the Watana reservoir is estimated to be 85, and at Devil Canyon 55 feet. However, during the driest sequence of simulated riverflows, Watana would be drawndown to its optimum minimum level of 2,045 feet (see Section 12.11).

(c) Energy Potential of the Watana-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 annual energy potential has been made to derive the firm annual energy potential or the dependable capacity of the hydro development.

The Federal Energy Regulatory Commision (FERC) in their publication Hydroelectric Power Evaluation (DOE/FERC-0031 of August 1979) 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". As described in Section 7.2, the recorded lowest flow in the Susitna river at Gold Creek has a recurrence frequency of the order of 1 in 10,000 years. This is considered an extremely rare event to be considered in an electrical system reliability evaluation. The critical streamflow sequence of record for the reservoir operation simulation is the 32-month period between October 1967 and May 1970, resulting in extreme drawdown of reservoirs in 1970 and 1971. This sequence has a recurrence interval of 1 in 300 years. Based on the Railbelt system studies and previous experience on large hydroelectric projects, it was assumed that a recurrence period of the order of 1:40 to 1:50 years dry hydrological sequence would constitute an adequate reliability for the 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 15.4). 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 again a conservative interpretation of the FERC definition.

The monthly distribution of firm annual energy as simulated in the reservoir operation has been used in system generation planning studies. Average monthly energy based on the recorded sequence hydrology is used in the economic analysis (Section 18.1).

(d) Reservoir Filling Sequence

Given the relative sizes of the Watana and Devil Canyon reservoirs, it is apparent that the most significant impact on the downstream flow regime will occur during filling of the Watana reservoir. Since this will be the first reservoir filled, careful planning is essential.

(i) Watana Reservoir Impoundment

Minimum monthly flows that must be maintained in the river below the dam during filling were established in consultation with fisheries and other environmental study groups and agencies. Table 15.2 presents the minimum monthly flow that is considered acceptable for river maintenance and fisheries requirements during the filling period. With the above minimum flow requirement, it would take at least 2-1/2 years of average stream flow to fill the reservoir.

Other major considerations in determination of the reservoir filling sequence is the level of the fill dam construction, available flood discharge facilities and the permissible risk of overtopping the partially constructed dam during unusual floods in the river. It has been assumed that a minimum storage volume will be maintained behind the partially constructed dam at all times during the filling period so that with available discharge facilities (low level outlets and service outlet works as they become available) a 1 in 100 year flood could be safely absorbed without overtopping of the dam. This figure has been selected as acceptable on the assumption that short-term flood forecasting will be made during the filling period. Consequently, with careful monitoring of snow pack in the basin and storm tracking, potentially damaging streamflows could be predicted with sufficient warning to lower the reservoir level in time.

It may be noted that the placement of the fill dam critically controls the reservoir filling in average streamflow years and restricts earlier filling should wet years be experienced. The driest recorded streamflow sequence would extend the filling period by one year.

The filling sequence in the years of average streamflow would allow first power on line by July, 1993. The units could be tested and commissioned prior to this date. A bonus in power and energy could be gained with one or two units installed by July 1992 when the power intake will be submerged sufficiently to allow power generation utilizing the minimum downstream discharge required.

(ii) Devil Canyon Reservoir

With Watana Reservoir in operation, the filling of the Devil Canyon Reservoir is relatively easily accomplished. Average monthly power flows from Watana between the months October through December in a single year will fill the reservoir while maintaining the minimum downstream flow requirements (see Table 15.2).

(e) Operating Capabilities of Susitna Units

(i) Turbine Performance

The reservoir operation studies described above, show that the Watana plant output may vary anywhere from zero with the unit at standstill on spinning reserve, to 1,200 MW when the six units are operating under maximum output at maximum head. (Note that there is a limitation in loading of a single unit in the rough zone of turbine operation from above speed-no-load operation to about 50 percent load). The four units at Devil Canyon have a maximum total output of 700 MW at maximum head.

The operating conditions of the turbines determining its characteristics are summarized in Table 15.3.

The turbine design head corresponds to the weighted average head. Based on the predicted daily load curves through the year 2010 and expected reservoir operation, it is expected that each unit at Watana is to supply a load averaging between 196 MW and 100 MW. This is the load which corresponds most closely to the best efficiency operation of the turbine.

Similarly the Devil Canyon units will supply a load between 174 MW and 100 MW.

(ii) Expected Unit Performance Characteristics

The rated output of the turbine corresponds to full gate operation at the rated head. Each turbine should operate satisfactorily at the maximum head. The output of the generator is limited by its continuous maximum rating of 115 percent with a maximum temperature rise of 80°C. The continuous maximum rating of the generator determines the maximum output of the unit and it will be necessary to limit the turbine output to this value accordingly at higher heads.

The expected plant performance at varying heads between minimum and maximum heads is shown in Figures 12.22 and 12.23. The plant efficiency with different number of units in operation is shown in Figures 15.2 and 15.3. In practice, for Watana, the load following requirements of the plant results in widely varying loading and resulting efficiency of operation.

(iii) Stability and Governing of Units

The required flywheel effect (inertia) of the unit is governed by the stability requirements of the units, namely the stabilization of the frequency for small load fluctuations. The machine inertia also influences the transient stability of the units during transmission line tripout following electrical faults in the system, a larger machine inertia decreasing the initial swing of the generators. On the other hand, a larger machine inertia decreases the natural frequency of oscillation of the machine, and increases the possibility of resonance with hydraulic surges in the draft tube and penstock.

Electrical transient stability studies of the Railbelt system indicate that the "natural" inertia of 3.2 to 3.5 kW-sec/KVA for the Watana and Devil Canyon generators is adequate for electrical stability of the system.

The pertinent plant data for stability and governing are given in Sections 12 and 13 in Watana and Devil Canyon plant respectively.

Pressure rise and speed rise are within normally acceptable limits of about 40 to 50 percent. A low ratio of the starting time of the water masses to the mechanical starting time of the unit is an indication of the hydraulic stability and acceptable response (promptitude time constant) of the governor. Good governing response and stability is indicated for the Watana and Devil Canyon units, and is important from the overall considerations of system load following and load-frequency response of the units.

(f) Watana Plant Daily Simulation Studies

The objectives of the plant daily simulation studies are to present performance studies of the selected 6-170 MW unit plant at Watana. The studies demonstrate its improved performance in comparison with a 4-250 MW plant. The simulation program was arranged to:

- Study the operation and load following characteristics of the Watana powerplant with different number and rating of units;

- Determine the effect of minimum and maximum loading constraints of the units;

- Determine the effect of critical single or double contingency outages of units on the amount and type of spinning reserves available in the system;

- Study the effects of maintenance outages and its impact on generation scheduling and system security; and

- Check the operation of gas turbines and peaking plant.

(i) Computer Simulation Model

To achieve the stated objectives, a computer simulation program was used to simulate Watana power plant and system operation. The Watana turbines and reservoir are modeled in detail to simulate closely the reservoir regulation and load - following characteristics of the turbines.

The model includes the following principal features:

- Turbine characteristics as a function of head, gate opening (flow), and efficiency are used in the model.

- Minimum loading limitations of the turbine due to rough zone of operation up to 50 percent of the gate openings are constraints for turbine loading and operation.

- Maximum continuous rating (CMR) of the generators constitutes the maximum loading of the units. Higher turbine capability at higher heads is blocked at the generator CMR rating.

- Predicted daily system load demand curves are used for two typical load shapes for winter and summer, respectively. Monthly peak load variation of the load is taken into account.

- Reservoir characteristics as a function of level and storage. Average maximum and minimum reservoir levels are constraints for reservoir regulation and operation.

- Unit by unit loading and de-loading of Watana generators according to load demand (load-following) is done taking into account all constraints mentioned above. The program loads the units equally for maximum efficiency of operation.

- Loading steam plants as base-load plants, and gas turbines as peaking plants.

- Maintenance scheduling of the generating units.

(ii) Results of the Simulations

Printouts of the results of the simulations are included in Appendix ____. For each run, printouts are presented for the following outputs in a typical day in each month of the year 2000 (January to December):

- Watana plant kW output;
- Watana turbine kW output, with flow and efficiency for each unit;
- Watana turbine utilization, showing number of units loaded;
- Watana reservoir level;
- Peaking plant kW output;
- Total system load kW demand;
- Total system reserve, including maintenance outage;
- Watana reserve capacity; and
- Annual energy output of Watana, thermal plant, small hydro, gas turbine plants, and overall annual system energy.

Simulation result of a typical December, 2000 day is shown in Fig 15.6. The simulations indicate that the six unit Watana plant (6-170 MW) has superior overall performance in terms of load following, improved overall efficiency and minimum loading constraints of the units over the four unit plant (4-250 MW).

The overall reliability of the six unit Watana plant is better than the four unit plant. During maintenance the six unit plant has a planned outage of 170 MW, as opposed to 250 MW for the four unit plant. During peak December loading, a double contingency outage of two units brings down system reserve to 107 MW for the 6-170 MW unit plant and to less than zero for the 4-250 MW unit plant for the year 2000 on study.

The simulations indicate that sufficient spinning reserve of a minimum of one Watana unit is available for all peak day loadings for the six unit Watana plant for the year 2000 on study.

15.7 - Performance Monitoring

(a) Watana Dam

Instrumentation is installed to enable the performance of the dam to be monitored to ensure that its behavior is within the limits assumed in the design and to enable any variations beyond those limits to be recognized quickly so that remedial action can be taken without delay.

It is essential that continuous monitoring of the instrumentation installed in the dam be carried out by qualified personnel who thoroughly understand the significance of the readings and more importantly, the significance of variations in the readings.

The instrumentation is installed to monitor both short-term behavior during construction and initial filling and long-term behavior over the life of the dam. The short-term is the most significant period when the dam is subjected to its initial loading and the responsibility for reading the instruments, reducing the data and evaluating the results over that period normally rests with the design engineer. It is important that personnel who will ultimately be responsible for the monitoring are involved as early as possible in the development.

The most important aspects of the monitoring program and likely maintenance requirements are outlined below:

(i) Foundation Abutment Pore Pressures and Discharge From Drainage Pressure Relief System

Since sections of the foundation are frozen, the grouted cut-off may not be fully effective and leakage may increase as the rock temperature increases.

This condition would be indicated by increased discharge from the drainage system and would be remedied by additional grouting from the grouting gallery, possibly combined with additional drainage holes.

(ii) Quality of Discharge from Pressure Relief System

Any discoloration of the drainage system discharge would indicate the carry over of fine material either from the rock foundation or from the core. The problem area would be located and additional grouting carried out. Water quality should also be monitored for any change in material content.

(iii) Deformation of the Structure

Most deformation of the structure as observed by settlements and lateral movements is expected to occur soon after construction and under initial filling of the reservoir. Any excessive settlement would be made good to maintain freeboard. Deformation records would be correlated with such data as reservoir level, heavy storms and seismic activity.

(iv) Routine Observations

An essential part of any monitoring program is a regular routine visual inspection of all exposed parts of the structure and the area downstream of the dam for any unusual features such as local settlement or other movement, zones of seepage discharge, wet areas, and changes in vegetation. All exposed concrete surfaces would also be inspected and records kept of any signs of distress, cracking or deterioration.

(v) Relict Channel

Particular attention must be paid to monitoring the whole area of the relict channel, including regular readings of piezometers and thermistors of surface elevation survey monitoring and inspections of the discharge zone for changes in seepage flows and any signs of piping failure.

15.8 - Plant Operation and Maintenance

The system demand varies throughout the year from a winter (December/January) peak to a summer (July/August) trough, and from hour-to-hour throughout the day. The central dispatch center operates with the object of ensuring that sufficient plant is available at all times to meet the varying load in accordance with a merit-order schedule with due considerations to security.

On the other hand, generating plant must be maintained periodically for various reasons:

- Preventive maintenance, to ensure safe and reliable operation (performed either on load or shut down);
- Corrective maintenance, to restore lost efficiency of plant; and
- Emergency maintenance, arising from plant failure.

Stationary inspections of plant are also required.

To meet the conflicting requirements of supplying load and maintenance outages, the plant maintenance program is planned to fit into the electrical system generation program to determine the amount of plant which can be safely permitted to shut down for maintenance during each week of the year. Due consideration must be taken of such factors as:

- Repair capacity in maintenance workshops;
- Delivery of spares and materials;
- Availability of specialized labor, special equipment and similar resources;
- Expiration of statutory inspection periods;
- Weather conditions, for outdoor installation; and
- Concurrence of outages between generating plant and transmission system.

The plant at Watana and Devil Canyon are high merit-order plants and maintenance must be organized to minimize outages which affect availability. The expense and time of inspecting and maintaining the large and important machines at Watana and Devil Canyon in good condition, rather than operating the machines continuously until failure occurs in service can be justified on the basis of increased reliability and lower overall cost. With this in mind, sufficient spare capacity must be available in the Susitna plants and the system at all times to cover planned and forced outage of the large units.

(a) Frequency of Inspections and Maintenance

The degree of inspection at the plants varies from frequent periodic visual inspections to a complete major disassembly and thorough inspection at long intervals of about 10 to 15 years. Certain manufacturers and users recommend a major inspection after the first year of service. Factors influencing the degree of disassembly and period between inspections include the following:

- Findings at previous visual inspections;
- Results of previous tests;
- History of similar machines; and
- Frequency of starts, load cycling and overloading during service.

Ready availability and access of stored data on the plant computer system of the records of previous inspection and tests as well as current performance trends (such as, for example, abnormal temperature recordings) will improve overall maintenance and performance of the units.

Experience records from machines similar to the Watana and Devil Canyon machines indicate that a minimum maintenance period of 5 to 6 days are required for each machine, resulting in an outage of 150 to 170 MW capacity for an average period of 50 to 60 days in the years. In exceptional cases, certain machines may be down for greater maintenance periods. It is therefore reasonable to allow a total of 2-1/2 to 3 months planned outage as a conservative approach to system generation and maintenance planning for the Susitna units. In principle, these outages are scheduled during the months of June to August when the lower summer load demands make it possible to release the units for maintenance. The actual outages will be coordinated on a week-to-week basis with the planned maintenance of the units in the rest of the system and will take into consideration emergency shutdowns, breakdowns, delays in construction and maintenance and other unforeseen contingencies.

(b) Access and Maintenance in the Powerhouse

Techniques developed both in the design and the operation of conventional underground hydroelectric powerplants have resulted in underground facilities which are not significantly more difficult to maintain than surface plants. Isolation of underground installations from both penstock water and from tailrace water is a vitally important factor. Downstream water conduits with manifolds require draft tube isolating devices of appropriate design. Drainage and dewatering facilities must be highly reliable and of adequate capacity.

There will be situations where a decision must be made as to whether to carry out maintenance and repair work on components underground or on the surface. Many items are large and heavy and therefore are best handled by the powerhouse crane. Sufficient erection bay space and laydown area between the generating units are provided for all normal maintenance and overhaul needs.

Transformers will be moved within the access tunnel and transformer gallery by means of wheels mounted on the transformer base.

The greatest demand in laydown space within the powerhouse cavern is likely to occur during the initial equipment installation process and the 10 to 15 year major disassembly/maintenance. The working area will be sized to allow the simultaneous placing of turbine and generator components.

Adequate crane facilities are provided both for installation and maintenance. The main powerhouse overhead cranes have a capacity of about 200 tons and will be equipped with a 20-ton auxiliary hoist. Small monorail hoists will be provided as necessary at intermediate levels within powerplant caverns for handling equipment likely to require movement for inspection, maintenance and/or replacement.

(c) Major Overall Activities

The major activities which require special space and handling consideration in the plants include:

- Replacing generator stator winding coils	--performed in situ, may require removal of rotor assembly.
- Rotor inspection	--performed for losseness, overheating or short circuits, specially after a major trip out causing full overspeed.
- Replacement of thrust-bearing assemblies	--designs normally permit removal and replacement of components without major dismantling, but at major overhaul intervals removal and strip down will be advisable.
- Replacement of runner seals	--requires dismantling and removal of runner component. This may be possible from below by removing the draft tube cone and bottom cover, or alternatively from above by removing the head cover and runner.
- Cavitation damage repair to runner/impeller	--will normally involve access through draft tube for inspection and minor repair, but ultimately requires runner removal as outlined above.
- Repair and refinishing of water passage steel and concrete surfaces	--access to the penstock will occur at about 5 to 6 year intervals. Major painting or refinishing will probably not be required until 10 to 12 years after commisioning. More frequent access will be provided to the downstream water passages when isolated by the draft tube gates and unwatered by the station dewatering system.

- Generator circuit breaker repair	--particular attention and a high level of maintenance are required for generator circuit breakers.
- Transformer maintenance	--general maintenance will be carried out in situ in the transformer gallery. Major overhaul or repair requiring untaking of transformer windings will be done in the powerhouse erection bay with adequate crane facilities.

(d) Maintenance Workshops and Operating/Maintenance Staff

The Watana and Devil Canyon powerplant are each provided with workshops to facilitate the normal maintenance needs of each plant. The workshop block includes operations for fitting and machining, welding, electrical, and relay instrumentation, with adequate stores for tools and spare parts. The Watana powerplant will be provided additionally with surface maintenance and central storage facilities to cater to the needs of both plants.

Maintenance operation plannings of both plants are centralized at Watana. Staff will be normally located at Watana and housed at the operators village at Watana. With centralized control of the Susitna project located at Watana, the Devil Canyon plant will not have a resident operating and maintenance staff. Proper road and transport facilities should be maintained between Watana and Devil Canyon to facilitate movement of personnel and/or equipment between the plants.

The central maintenance staff should include the following recommended minimum personnel:

1 - superintendent of maintenance
1 - electrical maintenance engineer
1 - mechanical (and building) maintenance engineer
1 - instrumentation maintenance engineer
6 - assistant maintenance engineers; at peak maintenance will work on a shift basis

Both the Watana and Devil Canyon power plants are designed to be normally operated from the Susitna Area Control Center at Watana. The operating staff will be stationed at Watana and would consist of the following personnel:

1 - superintendent, Susitna project operations
1 - chief operator
3 - control room operators (on shift basis)
2 - powerhouse operators
2 - assistant powerhouse operators
2 - computer system operators

When necessary, operators will travel to Devil Canyon to assist in operation and/or maintenance programs at the powerplant.

TABLE 15.1: 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

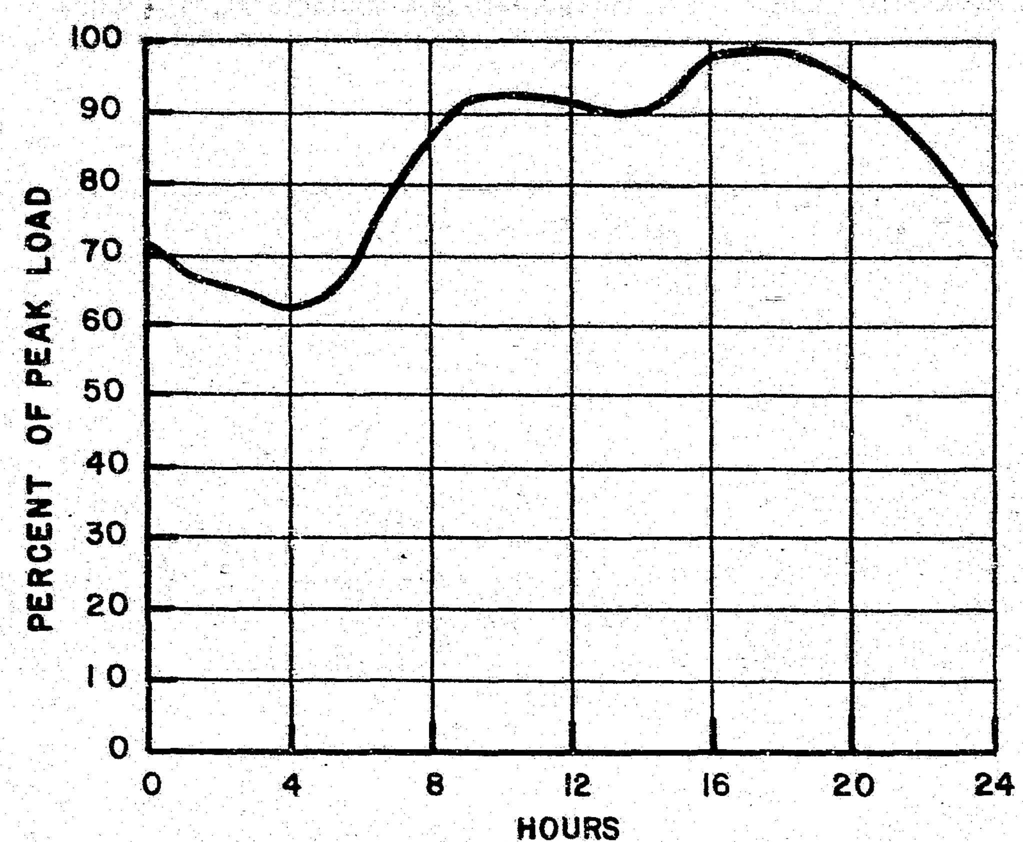
NOTE: Cases B and C were similar and only Case C was analyzed in detail.

TABLE 15.2: MINIMUM ACCEPTABLE FLOWS BELOW WATANA DAM DURING RESERVOIR FILLING

MONTH	MINIMUM ACCEPTABLE FLOW CFS
OCT	2050
NOV	900
DEC	900
JAN	900
FEB	900
MAR	900
APR	900
MAY	4000
JUN	4000
JUL	6000
AUG	6000
SEP	4600

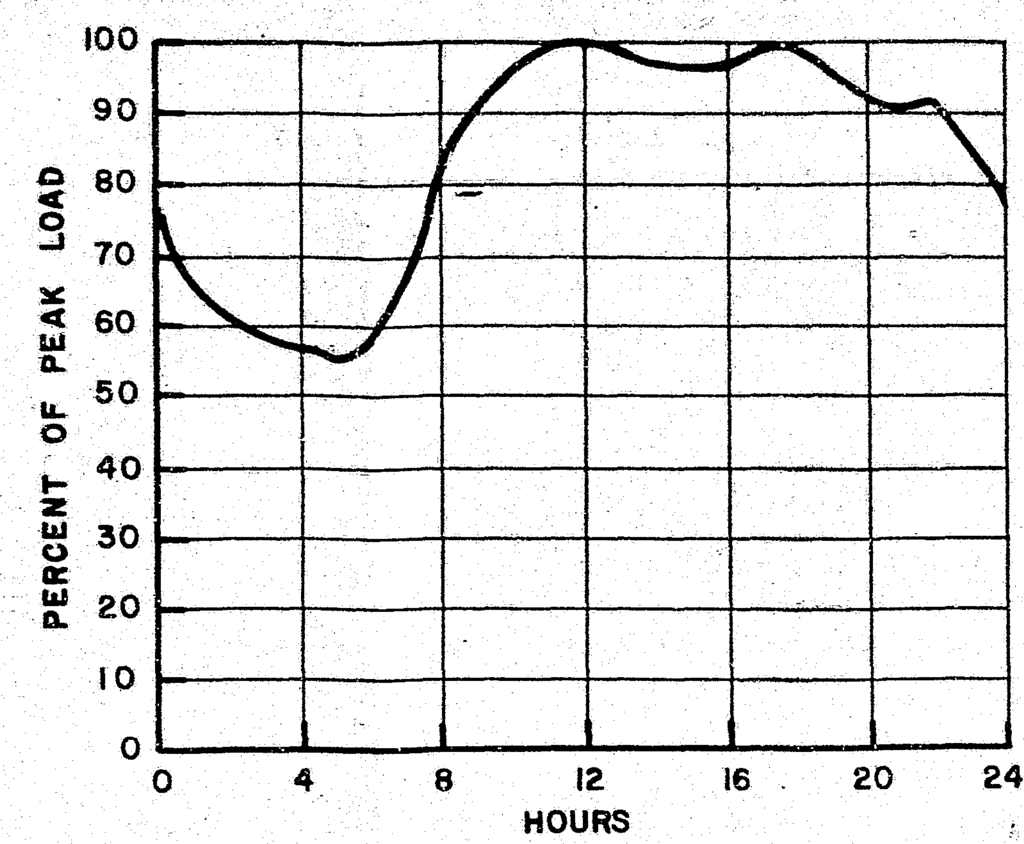
TABLE 15.3: TURBINE OPERATING CONDITIONS

	Watana	Devil Canyon
Maximum net head	725 feet	597 feet
Minimum net head	580 feet	238 feet
Design head	680 feet	575 feet
Rated head	680 feet	575 feet
Turbine flow at rated head	3550 feet	3800 feet
Turbine efficiency at design head	91%	91%
Turbine-generating rating at rated head	181,500 kW	164,000 kW



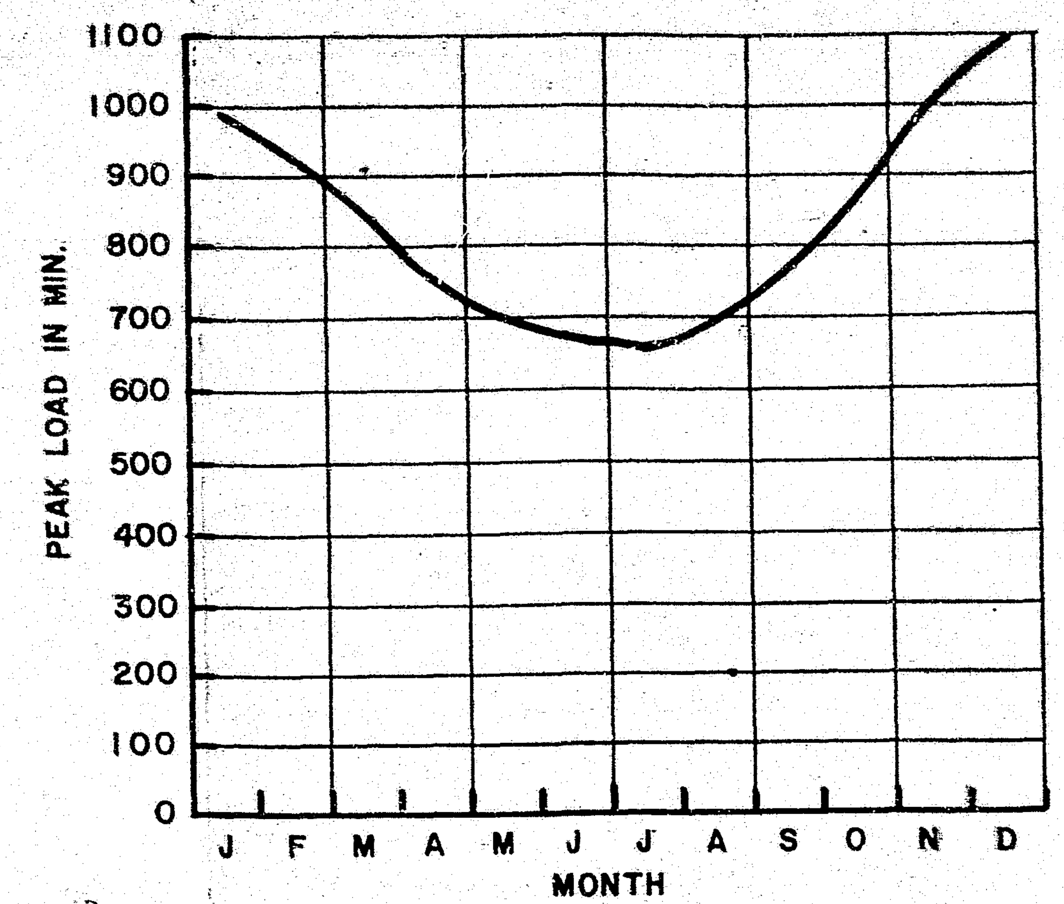
WINTER WEEKDAY
HOURLY LOAD VARIATION

NOTE: PEAK MW DECEMBER 2000 AD = 1084 MW



SUMMER WEEKDAY
HOURLY LOAD VARIATION

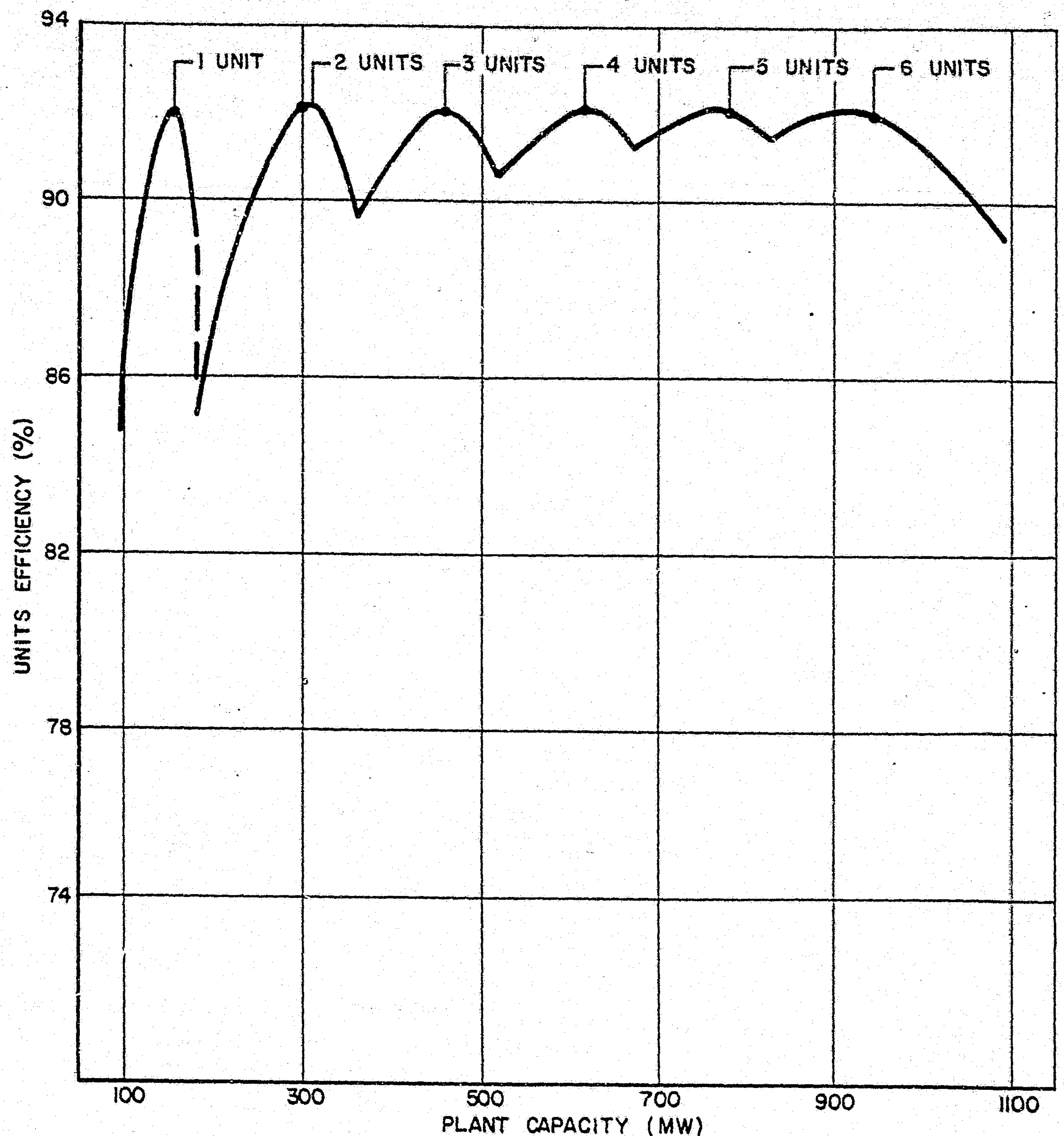
NOTE: PEAK MW JULY 2000 AD = 658 MW



LOAD VARIATION
IN YEAR 2000

TYPICAL LOAD VARIATION
IN ALASKA RAILBELT SYSTEM

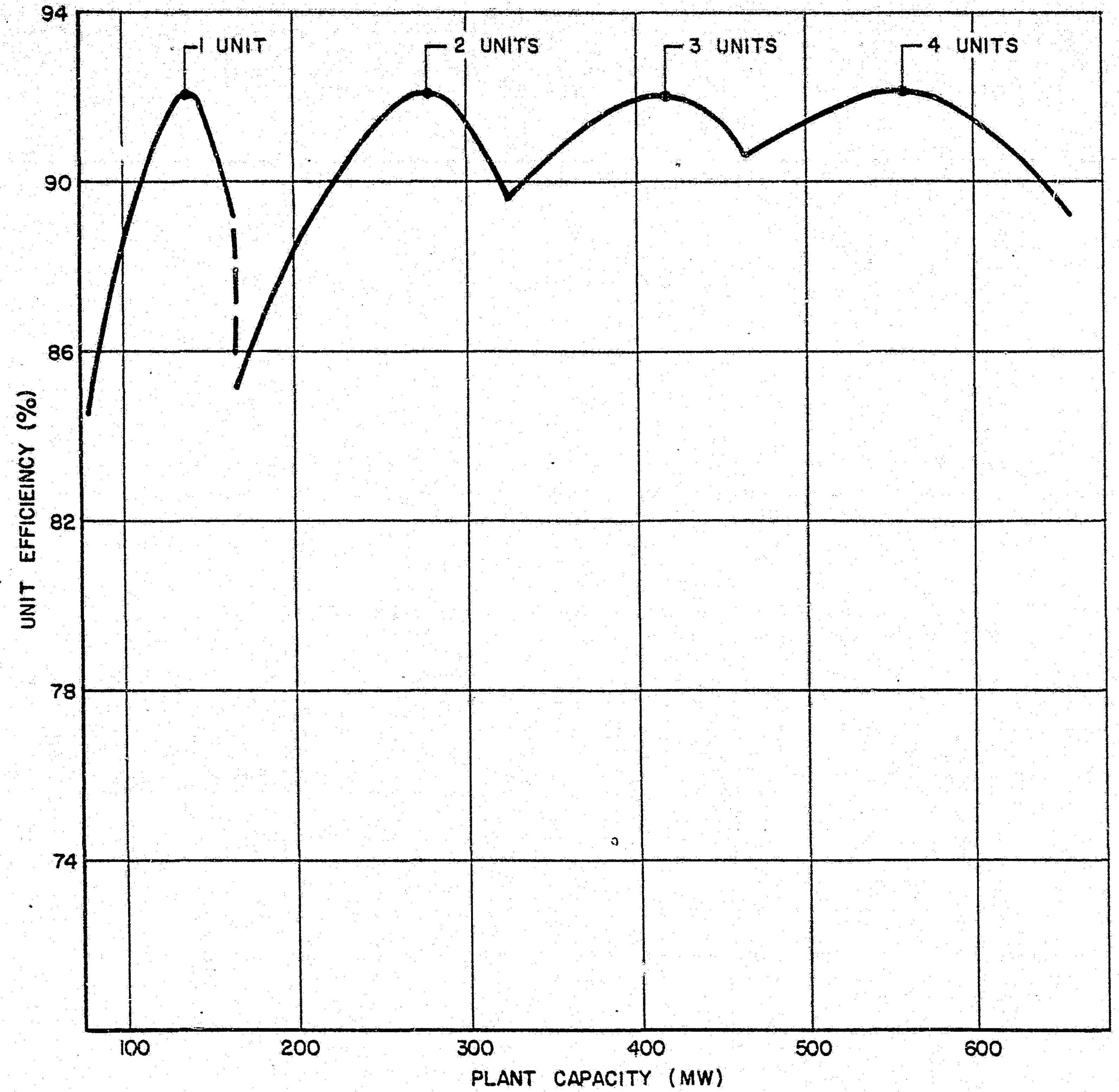
FIGURE 15.1



WATANA-UNIT EFFICIENCY
(AT RATED HEAD)

ACRES

FIGURE 15.2



DEVIL CANYON - UNIT EFFICIENCY
(AT RATED HEAD)

FIGURE 15.3

ACRES

SUSITNA PROJECT SIMULATION : 2000(MED.LOAD): WATANA 6-170 Page 6

WATANA verses TIME — Minimum 1.0821E+05 — Maximum 8.0387E+05

RESERV verses TIME — Minimum 4.4711E+05 — Maximum 1.1428E+06

	TIME	WATANA	N	GENKW	EFF	RESERV	INSTAL	KWLOAD	WATRES
TYPICAL NOVEMBER DAY: 16 HRS.	1.0625E+01	6.4956E+05	4.0000E+00	1.6239E+05	9.1580E-01	6.0144E+05	1.5310E+06	9.2956E+05	4.2646E+05
	1.0667E+01	6.8468E+05	4.0000E+00	1.7117E+05	9.0705E-01	5.6632E+05	1.5310E+06	9.6468E+05	3.9129E+05
18	1.0708E+01	7.1935E+05	5.0000E+00	1.4387E+05	9.1747E-01	5.3165E+05	1.5310E+06	9.9935E+05	3.5656E+05
	1.0750E+01	7.0940E+05	4.0000E+00	1.7735E+05	8.9866E-01	5.4160E+05	1.5310E+06	9.8940E+05	3.6645E+05
20	1.0792E+01	6.8608E+05	4.0000E+00	1.7152E+05	9.0661E-01	5.6492E+05	1.5310E+06	9.6608E+05	3.8971E+05
	1.0833E+01	6.6276E+05	4.0000E+00	1.6569E+05	9.1455E-01	5.8824E+05	1.5310E+06	9.4276E+05	4.1298E+05
22	1.0875E+01	6.3935E+05	4.0000E+00	1.5984E+05	9.1651E-01	6.1165E+05	1.5310E+06	9.1935E+05	4.3634E+05
	1.0917E+01	5.7272E+05	4.0000E+00	1.4318E+05	9.1688E-01	6.7828E+05	1.5310E+06	8.5272E+05	5.0292E+05
24	1.0958E+01	5.0609E+05	3.0000E+00	1.6870E+05	9.1049E-01	7.4491E+05	1.5310E+06	7.8609E+05	5.6951E+05
TYPICAL DECEMBER DAY: 0 HRS.	1.1000E+01	4.8959E+05	3.0000E+00	1.6320E+05	9.1560E-01	7.6141E+05	1.5310E+06	7.6959E+05	5.8598E+05
2	1.1042E+01	4.6429E+05	3.0000E+00	1.5476E+05	9.1789E-01	7.8671E+05	1.5310E+06	7.4429E+05	6.1123E+05
	1.1083E+01	4.3900E+05	3.0000E+00	1.4633E+05	9.1944E-01	8.1200E+05	1.5310E+06	7.1900E+05	6.3649E+05
4	1.1125E+01	4.1381E+05	3.0000E+00	1.3794E+05	9.1258E-01	8.3719E+05	1.5310E+06	6.9381E+05	6.6165E+05
	1.1167E+01	4.3549E+05	3.0000E+00	1.4516E+05	9.1847E-01	8.1551E+05	1.5310E+06	7.1549E+05	6.3994E+05
6	1.1208E+01	4.5717E+05	3.0000E+00	1.5239E+05	9.1854E-01	7.9383E+05	1.5310E+06	7.3717E+05	6.1822E+05
	1.1250E+01	4.7897E+05	3.0000E+00	1.5966E+05	9.1657E-01	7.7203E+05	1.5310E+06	7.5897E+05	5.9638E+05
8	1.1292E+01	5.5847E+05	4.0000E+00	1.3962E+05	9.1393E-01	6.9253E+05	1.5310E+06	8.3847E+05	5.1684E+05
	1.1333E+01	6.3796E+05	4.0000E+00	1.5949E+05	9.1662E-01	6.1304E+05	1.5310E+06	9.1796E+05	4.3730E+05
10	1.1375E+01	7.1728E+05	5.0000E+00	1.4346E+05	9.1706E-01	5.3372E+05	1.5310E+06	9.9728E+05	3.5792E+05
	1.1417E+01	7.1728E+05	5.0000E+00	1.4346E+05	9.1705E-01	5.3372E+05	1.5310E+06	9.9728E+05	3.5786E+05
NOON	1.1458E+01	7.1728E+05	5.0000E+00	1.4346E+05	9.1704E-01	5.3372E+05	1.5310E+06	9.9728E+05	3.5780E+05
	1.1500E+01	7.1729E+05	5.0000E+00	1.4346E+05	9.1704E-01	5.3371E+05	1.5310E+06	9.9729E+05	3.5773E+05
14	1.1542E+01	7.2090E+05	5.0000E+00	1.4418E+05	9.1762E-01	5.3010E+05	1.5310E+06	1.0009E+06	3.5405E+05
	1.1583E+01	7.2451E+05	5.0000E+00	1.4490E+05	9.1820E-01	5.2649E+05	1.5310E+06	1.0045E+06	3.5038E+05
16	1.1625E+01	7.2821E+05	5.0000E+00	1.4564E+05	9.1880E-01	5.2279E+05	1.5310E+06	1.0082E+06	3.4662E+05
	1.1667E+01	7.6630E+05	5.0000E+00	1.5326E+05	9.1833E-01	4.8470E+05	1.5310E+06	1.0463E+06	3.0947E+05
18	1.1708E+01	8.0389E+05	5.0000E+00	1.6078E+05	9.1629E-01	4.4711E+05	1.5310E+06	1.0839E+06	2.7081E+05
	1.1750E+01	7.9310E+05	5.0000E+00	1.5862E+05	9.1688E-01	4.5790E+05	1.5310E+06	1.0731E+06	2.8153E+05
20	1.1792E+01	7.6781E+05	5.0000E+00	1.5356E+05	9.1826E-01	4.8319E+05	1.5310E+06	1.0478E+06	3.0675E+05
	1.1833E+01	7.4252E+05	5.0000E+00	1.4850E+05	9.1963E-01	5.0848E+05	1.5310E+06	1.0225E+06	3.3198E+05
22	1.1875E+01	7.1711E+05	5.0000E+00	1.4342E+05	9.1695E-01	5.3389E+05	1.5310E+06	9.9711E+05	3.5732E+05
	1.1917E+01	6.4405E+05	4.0000E+00	1.6121E+05	9.1619E-01	6.0615E+05	1.5310E+06	9.2485E+05	4.2953E+05
24 HRS.	1.1958E+01	5.7250E+05	4.0000E+00	1.4315E+05	9.1671E-01	6.7842E+05	1.5310E+06	8.5258E+05	5.0174E+05

WATANA PLANT OUTPUT (KW)

NUMBER OF WATANA UNITS IN OPERATION

WATANA GENERATOR OUTPUT (KW, EACH UNIT)

UNIT EFFICIENCY (PERCENT)

SYSTEM RESERVE CAPACITY (KW)

SYSTEM INSTALLED CAPACITY (KW)

SYSTEM TOTAL LOAD (KW)

WATANA PLANT RESERVE (KW)

WATANA PLANT SIMULATION
DECEMBER 2000

FIGURE 15.4

ACRES

16 - ESTIMATES OF COST

This section presents estimates of capital and operating costs for the Susitna Hydroelectric Project, comprising the Watana and Devil Canyon developments and associated transmission and access facilities. The costs of design features and facilities incorporated into the project to mitigate environmental impacts during construction and operation are identified. A cash flow schedule, outlining capital requirements during planning, construction, and start-up is presented. The section also includes estimates of the cost of capital funds required during construction, based on alternative financing scenarios. The approach to the derivation of the capital and operating costs estimates is described.

The total cost of the Watana and Devil Canyon projects is summarized in Table 16.1. A more detailed breakdown of cost for each development is presented in Tables 16.2 and 16.3.

16.1 - Construction Costs

This section describes the process used for derivation of construction costs and discusses the Code of Accounts established, the basis for the estimates and the various assumptions made in arriving at the estimates. For general consistency with planning studies, all costs developed for the project are in January, 1982 dollars.

(a) Code of Accounts

Estimates of construction costs were developed using the FERC format as outlined in the Federal Code of Regulations, Title 18 (1).

The estimates have been subdivided into the following main cost groupings:

Group	Description
Production Plant	Costs for structures, equipment, and facilities necessary to produce power.
Transmission Plant	Costs for structures, equipment, and facilities necessary to transmit power from the sites to load centers.
General Plant	Costs for equipment and facilities required for the operation and maintenance of the production and transmission plant.
Indirect Costs	Costs that are common to a number of construction activities. For this estimate only camps' and electric power costs have been included in this group. Other indirect costs have been included in the costs under production, transmission, and general plant costs.

Overhead Construction Costs	Costs for engineering and administration.

Further subdivision within these groupings was made on the basis of the various types of work involved, as typically shown in the following example:

- Group:	Production Plant
- Account 332:	Reservoir, Dam, and Waterways
- Main Structure 332.3:	Main Dam
- Element 332.31:	Main Dam Structure
- Work Item 332.311:	Excavation
- Type of Work:	Rock

The detailed schedule of account items is presented in Appendix F.

(b) Approach to Cost Estimating

The estimating process used generally included the following steps:

- Collection and assembly of detailed cost data for labor, material, and equipment as well as information on productivity, climatic conditions, and other related items;

- Review of engineering drawings and technical information with regard to construction methodology and feasibility;

- Production of detailed quantity takeoffs from drawings in accordance with the previously developed Code of Accounts and item listing;

- Determination of direct unit costs for each major type of work by development of labor, material, and equipment requirements; development of other costs by use of estimating guides, quotations from vendors, and other information as appropriate;

- Development of construction indirect costs by review of labor, material equipment, supporting facilities, and overheads; and

- Development of construction camp size and support requirements from the labor demand generated by the construction direct and indirect costs.

The above steps are discussed in detail in the following:

(c) Cost Data

Cost information was obtained from standard estimating sources, from sources in Alaska, from quotes by major equipment suppliers and vendors, and from representative recent hydroelectric projects. Labor and equipment costs for 1982 were developed from a number of sources (1,2,3) and from an analysis of costs for recent projects performed in the Alaska environment.

It has been assumed that contractors will work an average of two 9-hour shifts per day, 6 days per week, with an expected range as follows:

Mechanical/Electrical Work	8-hour shifts
Formwork/Concrete Work	9-hour shifts
Excavation/Fill Work	10-hour shifts

These assumptions provide for high utilization of construction equipment and reasonable levels of overtime earnings to attract workers. The two-shift basis generally achieves the most economical balance between labor and camp costs.

Construction equipment costs were obtained from vendors on an FOB Anchorage basis with an appropriate allowance included for transportation to site. A representative list of construction equipment required for the project was assembled as a basis for the estimate. It has been assumed that most equipment would be fully depreciated over the life of the project. For some activities such as construction of the Watana main dam, an allowance for major overhaul was included rather than fleet replacement. Equipment operating costs were estimated from industry source data, with appropriate modifications for the remote nature and extreme climatic environment of the site. Fuel and oil prices have also been included based upon FOB site prices.

Information for permanent mechanical and electrical equipment was obtained from vendors and manufacturers who provided guideline costs on major power plant equipment.

The costs of materials required for site construction were estimated on the basis of suppliers' quotations, adjusted for Alaskan conditions.

(d) Seasonal Influences on Productivity

A review of climatic conditions, together with an analysis of experience in Alaska and in Northern Canada on large construction projects was undertaken to determine the average duration for various key activities. The results of this evaluation are presented in Table 16.4 and these durations have been used to develop the construction cost estimates.

In general, it has been conservatively assumed for current study purposes that a 10-month construction season is the maximum feasible duration and that most work operations will cease during December and January because of the extreme cold weather and the associated lower productivity. Productivity is assumed to decrease by 30 percent during the November, February, and March time periods. This productivity decrease results from a combination of low temperatures, reduced daylight hours, precipitation, and soil conditions.

Studies by others (4) have indicated a 60 percent or greater decrease in efficiency in earthwork operations under such adverse conditions. Typical efficiency curves for the Fairbanks location, which were used as a guide, are shown in Figure 16.1. Although this curve cannot be used directly for the Susitna Hydroelectric Project, it illustrates the relative seasonal effect on manual labor, hauling, and earth excavation that is likely to be experienced.

Studies performed as part of this work program indicate that the general construction productivity at the Susitna damsite during the months of April through September would be comparable with that in the northern sections of the Western United States.

Rainfall in the general region of the site is moderate between mid-April and mid-October ranging from a low of 0.75 inches precipitation in April to a high of 5.33 inches in August. This moderate amount of rainfall should not create significant problems during fill placement activities because of the good quality river run borrow material used in the haul road and dam embankment. Temperatures in this period range from 33° to 66° for a twenty-year average. In the five-month period from November through March, the temperature ranges from 9.4°F to 20.3°F with snowfall of 10 inches per month. Excavation of gravel from the river or below ground water level and placing the material in a fill would have to be discontinued during these months because of the snow and ice intrusion and the inability to obtain satisfactory compaction with frozen material. However, other construction activities could continue during this period (consideration being given to the cost of snow removal) with possibly the exception of the two coldest months of December (9.6°F) and January (9.4°F).

Productivity would decrease 10 percent to 30 percent from the normal or base production rates during the periods of mid-October through November, and February through mid-April, and up to 50 percent during December and January depending upon the activity and the exposure.

(e) Construction Methods

The construction methods assumed for development of the estimate and construction schedule, are generally considered as "normal", in line with the available level of technical information. A conservative approach has been taken in those areas where more detailed information will be developed during subsequent investigation and engineering programs. For example, normal drilling, blasting, and mucking methods have been assumed for all underground excavation. Also conventional equipment has been considered for major fill and concrete work. Various construction methods were considered for several of the major work items to determine the most economically practical method. For example, a comprehensive evaluation was made of the means of excavating material from Borrow Area E and the downstream river for the Watana dam shells. A comparison of excavation by dragline, dredge, backhoe, and "sauerman" (scraper) bucket methods was made, with consideration given to the quantity of material available, distance from the dam, and location in the river or adjacent terraces.

(f) Quantity Takeoffs

Detailed quantity takeoffs were produced from the engineering drawings using methods normal to the industry. The quantities developed are those listed in the detailed summary estimates in Appendix F.

(g) Indirect Construction Costs

Indirect construction costs were estimated in detail for the civil construction activities. A more general evaluation was used for the mechanical and electrical work.

Indirect costs included the following:

- Mobilization;
- Technical and supervisory personnel above the level of trades foremen;
- All vehicle costs for supervisory personnel;
- Fixed offices, mobile offices, workshops, storage facilities, and lay-down areas, including all services;
- General transportation for workmen on site and off site;
- Yard cranes and floats;
- Utilities including electrical power, heat, water, and compressed air;
- Small tools;
- Safety program and equipment
- Financing;
- Bonds and securities;
- Insurance;
- Taxes;
- Permits;
- Head office overhead;
- Contingency allowance; and
- Profit.

16.2 - Mitigation Costs

As discussed in previous sections, the project arrangement includes a number of features designed to mitigate potential impacts on the natural environment and on residents and communities in the vicinity of the project. In addition a number of measures are planned during construction of the project to mitigate similar impacts caused by construction activities. The measures and facilities represent additional costs to the project than would be normally required for safe and efficient operation of a hydroelectric development. A summary of these mitigation costs is presented in Table 16.5. The costs include direct and indirect costs, engineering, administration, and contingencies.

A number of mitigation costs are associated with facilities, improvements or other programs not directly related to the project or located outside the project boundaries. These would include the following items:

- Caribou barriers;
- Fish channels;
- Fish hatcheries;
- Stream improvements;
- Salt licks;
- Recreational facilities;
- Habitat management for moose;
- Fish stocking program in reservoirs; and
- Land acquistion cost for recreation.

It is anticipated that some of these features or programs will not be required during or after construction of the project. In this regard a probability factor has been assigned to each of the above items, and the estimated cost of each reduced accordingly. The estimated cost of these measures, based on this procedure, is approximately $9 million. These costs have been assumed to be covered by the construction contingency.

A number of studies and programs will be required to monitor the impacts of the project on the environment and to develop and record various data during project construction and operation. These include the following:

- Archaeological studies;
- Fisheries and wildlife studies;
- Right-of-way studies; and
- Socioeconomic planning studies.

The costs for the above work have been estimated to be approximately $________ and included in the owner's costs under project overheads.

16.3 - Operation, Maintenance, and Replacement Costs

The facilities and procedures for operation and maintenance of the project are described in Section 15. Assumptions for the size and extent of these facilities have been conservatively made on the basis of experience at large hydroelectric developments in northern climates, noteably Canada. The annual costs developed for operation, maintenance, and interim replacement for the Watana and Devil Canyon projects and the transmission facilities are summarized in Tables 16._, 16._ and 16._.

16.4 - Engineering and Administration Costs

Engineering has been subdivided into the following accounts for the purposes of the cost estimates:

- Account 71

  . Engineering and Project Management
  . Construction Management
  . Procurement

- Account 76

  . Owner's Costs

The total cost of engineering and administrative activities has been estimated at 12.5 percent of the total construction costs, including contingencies. This is in general agreement with experience on projects similar in scope and complexity. A detailed breakdown of these costs is dependent on the organizational structure established to undertake design and management of the project, as well as more definitive data relating to the scope and nature of the various project components. However, the main elements of cost included are as follows:

(a) Engineering and Project Management Costs

These costs include allowances for:

- Feasibility studies, including site surveys and investigations and logistics support;

- Preparation of a license application to the FERC;
- Technical and administrative input for other federal, state and local permit and license applications;
- Overall coordination and administration of engineering, construction management, and procurement activities;
- Overall planning, coordination, and monitoring activities related to cost and schedule of the project;
- Coordination with APA and reporting to APA regarding all aspects of the project;
- Preliminary and detailed design;
- Technical input to procurement of construction services, support services, and equipment;
- Monitoring of construction to ensure conformance to design requirements;
- Preparation of start-up and acceptance test procedures; and
- Preparation of project operating and maintenance manuals.

(b) Construction Management Costs

Construction management costs have been assumed to include:

- Initial planning and scheduling and establishment of project procedures and organization;
- Coordination of onsite contractors and construction management activities;
- Administration of onsite contractors to ensure harmony of trades, compliance with applicable regulations, and maintenance of adequate site security and safety requirements;
- Development, coordination, and monitoring of construction schedules;
- Construction cost control;
- Material, equipment and drawing control;
- Inspection of construction and survey control;
- Measurement for payment;
- Start-up and acceptance test for equipment and systems;
- Compilation of as-constructed records; and
- Final acceptance.

(c) Procurement Costs

Procurement costs have been assumed to include:

- Establishment of project procurement procedures;
- Preparation of non-technical procurement documents;
- Solicitation and review of bids for construction services, support services, permanent equipment, and other items required to complete the project;
- Cost administration and control for procurement contracts; and
- Quality assurance services during fabrication or manufacture of equipment and other purchased items.

(d) Owner's Costs

Owner's costs have been assumed to include the following:

- Administration and coordination of project management and engineering organizations;
- Coordination with other state, local, and federal agencies and groups having jurisdiction or interest in the project;
- Coordination with interested public groups and individuals;
- Reporting to legislature and the public on the progress of the project; and
- Legal costs (Account 72).

16.5 - Allowance for Funds Used During Construction

At current high levels of interest rates in the financial market-place, AFDC will amount to a significant element of financing cost for the lengthy periods required for construction of the Watana and Devil Canyon projects. However, in economic evaluations of the Susitna project, the low real rates of interest assumed would have a much reduced impact on assumed project development costs. Furthermore, as discussed in Section 18, direct state involvement in financing of the Susitna project will also have a significant impact on the amount, if any, of AFDC. For purposes of the current feasibility study, therefore, the conventional practice of calculating AFDC as a separate line item for inclusion as part of project construction cost, has not been followed. Provisions for AFDC at appropriate rates of interest are made in the economic and financial analyses described in Section 18.

16.6 - Escalation

As noted, all costs presented in this Section are at January, 1982 levels, and consequently include no allowance for future cost escalation. Thus, these costs would not be truly representative of construction and procurement bid prices. This is because provision must be made in such bids for continuing escalation of costs, and the extent and variation of escalation which might take place over the lengthy construction periods involved. Economic and financial evaluations discussed in Section 18 take full account of such escalation at appropriately assumed rates.

16.7 - Cash Flow and Manpower Loading Requirements

The cash flow requirements for construction of Watana and Devil Canyon are an essential input to economic and financial planning studies discussed in Section 18. The basis for the cash flow are the construction cost estimates in January, 1982 dollars and the construction schedules presented in Section 17, with no provision being made as such for escalation. The results are presented in Figures 16.2 and 16.3. Similarly, the corresponding manpower loading requirements are shown in Figures 16.4 and 16.5. These curves were used as the basis for camp loading and associated socioeconomic impact studies.

LIST OF REFERENCES

(1) Code of Federal Regulations, Title 18, Conservation of Power and Water Resources, Parts 1 and 2, Washington, D.C., Government Printing Office, 1981.

(2) Handbook of Wages and Benefits for Construction Unions, January 1981, U. S. Department of Labor, Office of Construction Industry Services, 1981.

(3) Caterpillar Performance Handbook, Caterpillar Tractor Co., Peoria, Illinois, October 1981.

(4) Roberts, William S., Regionalized Feasibility Study of Cold Weather Earthwork, Cold Regions Research and Engineering Laboratory, July 1976, Special Report 76-2.

TABLE 16.1: SUMMARY OF COST ESTIMATE

Category	Watana	Devil Canyon	Total
	January 1982 Dollars $ X 10^6		
Production Plant	$1,969	$ 766	$2,745
Transmission Plant	388	91	479
General Plant	5	5	10
Indirect	449	222	671
Subtotal	$2,811	$ 1,094	$3,905
Contingency 17.5%	492	191	683
Total Construction	$3,303	$ 1,285	$4,588
Overhead Construction	413	161	574
TOTAL PROJECT	$3,716	$1,446	$5,162

TABLE 16.2

ACRES

ESTIMATE SUMMARY

WATANA

CLIENT	ALASKA POWER AUTHORITY
PROJECT	SUSITNA HYDROELECTRIC PROJECT
TYPE OF ESTIMATE	Preliminary
APPROVED BY	
JOB NUMBER	P5700.00
FILE NUMBER	P5700.06
SHEET	1 OF 5
BY	DATE
CHKD	DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	PRODUCTION PLANT			
330	Land & Land Rights	$ 51		
331	Powerplant Structures & Improvements	74		
332	Reservoir, Dams & Waterways	1,519		
333	Waterwheels, Turbines & Generators	65		
334	Accessory Electrical Equipment	21		
335	Miscellaneous Powerplant Equipment (Mechanical)	14		
336	Roads & Railroads	225		
	TOTAL PRODUCTION PLANT		$ 1,969	

TABLE 16.2

ESTIMATE SUMMARY

AGRES

WATANA

CLIENT: ALASKA POWER AUTHORITY

PROJECT: SUSITNA HYDROELECTRIC PROJECT

TYPE OF ESTIMATE: Preliminary

APPROVED BY:

JOB NUMBER: P5700.00

FILE NUMBER: P5700.06

SHEET 2 OF 5

BY: DATE:

CHKD: DATE:

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 1,969	
	TRANSMISSION PLANT			
350	Land & Land Rights	$ 8		
352	Substation & Switching Station Structures & Improvements	12		
353	Substation & Switching Station Equipment	129		
354	Steel Towers & Fixtures	130		
356	Overhead Conductors & Devices	99		
359	Roads & Trails	10		
	TOTAL TRANSMISSION PLANT		$ 388	
			$ 2,357	

TABLE 16.2

ESTIMATE SUMMARY

WATANA

CLIENT	ALASKA POWER AUTHORITY
PROJECT	SUSITNA HYDROELECTRIC PROJECT
TYPE OF ESTIMATE	Preliminary
APPROVED BY	
JOB NUMBER	P5700.00
FILE NUMBER	P5700.06
SHEET	3 OF 5
BY	DATE
CHKD	DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 2,357	
	GENERAL PLANT			
389	Land & Land Rights	$ -		Included under 330
390	Structures & Improvements	-		Included under 331
391	Office Furniture/Equipment			
392	Transportation Equipment			
393	Stores Equipment			
394	Tools Shop & Garage Equipment	5		
395	Laboratory Equipment			
396	Power-Operated Equipment			
397	Communications Equipment			
398	Miscellaneous Equipment			
399	Other Tangible Property			
	TOTAL GENERAL PLANT		$ 5	
			$ 2,362	

TABLE 16.2

ESTIMATE SUMMARY

WATANA

CLIENT	ALASKA POWER AUTHORITY
PROJECT	SUSITNA HYDROELECTRIC PROJECT
TYPE OF ESTIMATE	Preliminary
APPROVED BY	
JOB NUMBER	P5700.00
FILE NUMBER	P5700.06
SHEET	4 OF 5
BY	DATE
CHKD	DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 2,362	
	INDIRECT COSTS			
61	Temporary Construction Facilities	$ -		See Note
62	Construction Equipment	-		See Note
63	Camp & Commissary	449		
64	Labor Expense	-		
65	Superintendence	-		See Note
66	Insurance	-		See Note
69	Fees	-		See Note
	Note: Costs under accounts 61, 62, 64, 65, 66, and 69 are included in the appropriate direct costs listed above.			
	TOTAL INDIRECT COSTS		$ 449	
			$ 2,811	

TABLE 16.2

ESTIMATE SUMMARY

WATANA

ACRES

CLIENT: ALASKA POWER AUTHORITY
TYPE OF ESTIMATE: Preliminary

PROJECT: SUSITNA HYDROELECTRIC PROJECT
APPROVED BY:

JOB NUMBER: P5700.00
FILE NUMBER: P5700.06
SHEET 5 OF 5
BY ______ DATE ______
CHKD ______ DATE ______

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD (Construction Costs)		$ 2,811	
	Contingency 17.5%		492	
	TOTAL CONSTRUCTION COSTS		$ 3,303	
	OVERHEAD CONSTRUCTION COSTS (PROJECT INDIRECTS)			
71	Engineering/ Administration	$ 413		
72	Legal Expenses	-		Included in 71
75	Taxes	-		Not applicable
76	Administrative & General Expenses	-		Included in 71
77	Interest	-		Not included
80	Earnings/Expenses During Construction	-		Not included
	Total Overhead		413	
	TOTAL PROJECT COST		$ 3,716	

TABLE 16.3

ACRES

ESTIMATE SUMMARY

DEVIL CANYON

CLIENT: ALASKA POWER AUTHORITY

PROJECT: SUSITNA HYDROELECTRIC PROJECT

TYPE OF ESTIMATE: Preliminary

APPROVED BY:

JOB NUMBER: P5700.00

FILE NUMBER: P5700.06

SHEET 1 OF 5

BY: DATE:

CHKD: DATE:

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	PRODUCTION PLANT			
330	Land & Land Rights	$ 22		
331	Powerplant Structures & Improvements	72		
332	Reservoir, Dams & Waterways	571		
333	Waterwheels, Turbines & Generators	42		
334	Accessory Electrical Equipment	14		
335	Miscellaneous Powerplant Equipment (Mechanical)	12		
336	Roads & Railroads	43		
	TOTAL PRODUCTION PLANT		$ 776	

TABLE 16.3

ESTIMATE SUMMARY

DEVIL CANYON

CLIENT	ALASKA POWER AUTHORITY
PROJECT	SUSITNA HYDROELECTRIC PROJECT
TYPE OF ESTIMATE	Preliminary
APPROVED BY	
JOB NUMBER	P5700.00
FILE NUMBER	P5700.06
SHEET	2 OF 5
BY	DATE
CHKD	DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 776	
	TRANSMISSION PLANT			
350	Land & Land Rights	$ -		Included in Watana Estimate
352	Substation & Switching Station Structures & Improvements	7		
353	Substation & Switching Station Equipment	21		
354	Steel Towers & Fixtures	29		
356	Overhead Conductors & Devices	34		
359	Roads & Trails	-		Included in Watana Estimate
	TOTAL TRANSMISSION PLANT		$ 91	
			$ 867	

TABLE 16.3

ACRES	**ESTIMATE SUMMARY**	DEVIL CANYON	JOB NUMBER P5700.00
	CLIENT ALASKA POWER AUTHORITY	TYPE OF ESTIMATE Preliminary	FILE NUMBER P5700.06
	PROJECT SUSITNA HYDROELECTRIC PROJECT	APPROVED BY	SHEET 3 OF 5
			BY DATE
			CHKD DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 867	
	GENERAL PLANT			
389	Land & Land Rights	$ -		Included under 330
390	Structures & Improvements	-		Included under 331
391	Office Furniture/Equipment			
392	Transportation Equipment			
393	Stores Equipment			
394	Tools Shop & Garage Equipment	5		
395	Laboratory Equipment			
396	Power Operated Equipment			
397	Communications Equipment			
398	Miscellaneous Equipment			
399	Other Tangible Property			
	TOTAL GENERAL PLANT		$ 5	
			$ 872	

TABLE 16.3

ACRES	**ESTIMATE SUMMARY** DEVIL CANYON		JOB NUMBER P5700.00
	CLIENT ALASKA POWER AUTHORITY	TYPE OF ESTIMATE Preliminary	FILE NUMBER P5700.06
	PROJECT SUSITNA HYDROELECTRIC PROJECT	APPROVED BY	SHEET 4 OF 5
			BY DATE
			CHKD DATE

No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD		$ 872	
	INDIRECT COSTS			
61	Temporary Construction Facilities	$ -		See Note
62	Construction Equipment	-		See Note
63	Camp & Commissary	222		
64	Labor Expense	-		See Note
65	Superintendence	-		See Note
66	Insurance	-		See Note
69	Fees	-		See Note
	Note: Costs under accounts 61, 62, 64, 65, 66, and 69 are included in the appropriate direct costs listed above.			
	TOTAL INDIRECT COSTS		$ 222	
			$ 1,094	

TABLE 16.3

ACRES

ESTIMATE SUMMARY

DEVIL CANYON

CLIENT: ALASKA POWER AUTHORITY

PROJECT: SUSITNA HYDROELECTRIC PROJECT

TYPE OF ESTIMATE: Preliminary

APPROVED BY:

JOB NUMBER: P5700.00

FILE NUMBER: P5700.06

SHEET 5 OF 5

BY: DATE:

CHKD: DATE:

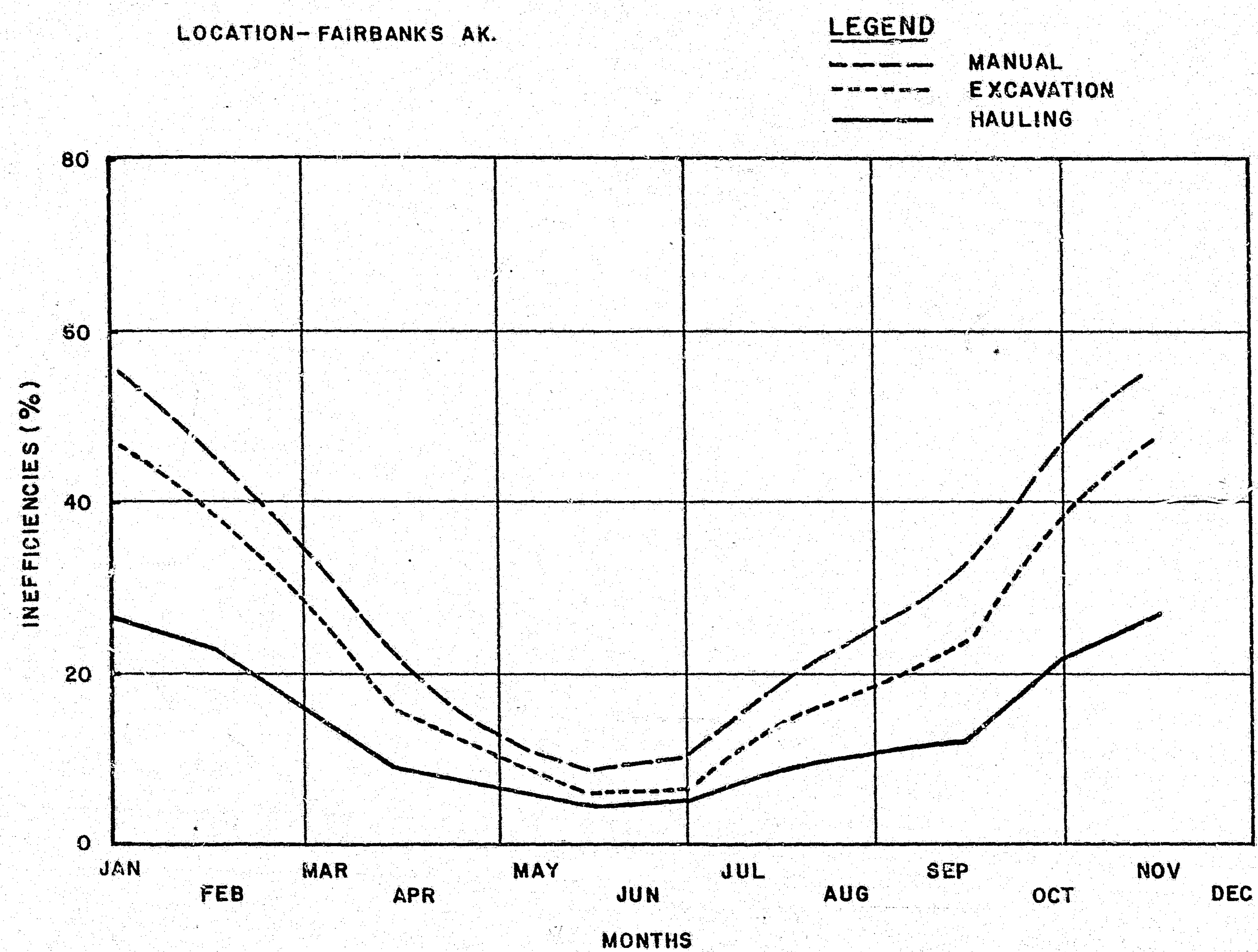
No.	DESCRIPTION	AMOUNT	TOTALS	REMARKS
	TOTAL BROUGHT FORWARD (Construction Costs)		$ 1,094	
	Contingency 17.5%		191	
	TOTAL CONSTRUCTION COSTS		1,285	
	OVERHEAD CONSTRUCTION COSTS (PROJECT INDIRECTS)			
71	Engineering	$ 161		
72	Legal Expenses	-		Included in 71
75	Taxes	-		Not Applicable
76	Administrative & General Expenses	-		Included in 71
77	Interest	-		Not Included
80	Earnings/Expenses During Construction	-		Not Included
	Total Overhead Costs		161	
	TOTAL PROJECT COST		$ 1,446	

TABLE 16.4: CONSTRUCTION SEASONS

WORK ITEM	DURATION (MONTHS)	DURATION (WORKING DAYS)	SEASON START/FINISH
Granular Fill Placement	6.0	150	Apr 15-Oct 15
Impervious Fill Placement	5.0	100	May 1-Oct 1
Rock Fill Placement	7.0	170	Apr 1-Dec 1
Underground Work	10.0	250	Feb 15-Dec 15
Concrete Placement (Generally)	8.0	200	Apr 1-Dec 1
Concrete Placement (Devil Canyon Dam)	10.0	250	Feb 15-Dec 15
Other Aboveground Work	10.0	250	Feb 15-Dec 15

TABLE 16.5: MITIGATION MEASURES SUMMARY OF COSTS INCORPORATED IN CONSTRUCTION COST ESTIMATES

COSTS INCORPORATED IN CONSTRUCTION ESTIMATES	$ X 10^3 WATANA	$ X 10^3 DEVIL CANYON	
Spillway Valves in Dam			
Main Dam at Devil Canyon		14,510	
Service Spillway at Watana	48,500		
Restoration of Borrow Area D	1,617		
Restoration of Borrow Area F	551		
Restoration of Camp	1,054	700	
Restoration of Construction Sites	4,050	2,016	
Fencing around Camps	414	305	
Fencing around Garbage Disposal Area	125	125	
Multilevel Intake Structure	18,000		
Camp Facilities Associated with trying to Keep Workers out of Local Communities	10,156	9,000	
Restoration of Haul Roads	756	505	
SUBTOTAL	85,221	27,161	
Contingency 17.5%	14,914	4,753	
TOTAL CONSTRUCTION	100,135	31,914	
Engineering 12.5%	12,517	3,989	
TOTAL PROJECT	112,542	35,903	148,555



EARTHWORK INEFFICIENCIES BASED ON MONTHLY TEMPERATURE, LIGHTING AND PRECIPITATION

FIGURE 16.1

ACRES

17 - DEVELOPMENT SCHEDULE

- To Follow

18 - ECONOMIC AND FINANCIAL EVALUATION

18.1 - Economic Evaluation

This section provides a discussion of the key economic parameters used in the study and develops the net economic benefits stemming from the Susitna Hydroelectric Project. Section 18.1(a) deals with those economic principles relevant to the analysis of net economic benefits and develops inflation and discount rates and the Alaskan opportunity values (shadow prices) of oil, natural gas and coal. In particular the analysis is focused on the longer term prospects for coal markets and prices. This follows from the evaluation that in the absence of Susitna, the next best thermal generation plan would rely on exploitation of Alaska coal. The future coal price is therefore considered in detail to provide rigorous estimates of prices in the most likely alternative markets and hence the market price of coal at the mine-head within the state.

Section 18.1(c) presents the net economic benefits of the proposed hydroelectric power investments compared with this thermal alternative. These are measured in terms of present valued differences between benefits and costs. Recognizing that even the most careful estimates will be surrounded by a degree of uncertainty, the benefit-cost assessments are carried out in a probabilistic framework. The analysis therefore provides both a most likely estimate of net economic benefits accruing to the state and a range of net economic benefits that can be expected with a likelihood (confidence level) of 95 percent or more.

(a) Economic Principles and Parameters

(i) Economic Principles - Concept of Net Economic Benefits

The concept of net economic benefits has a profound importance to the State of Alaska. A necessary condition for maximizing the increase in state income and economic growth is the selection of public or private investments with the highest present valued net benefits to the state. In the context of Alaskan electric power investments, the net benefits are defined as the difference between the costs of optimal Susitna-inclusive and Susitna-exclusive (all-thermal) generation plans.

The energy costs of power generation are initially measured in terms of opportunity values or shadow prices which may differ from accounting or market prices currently prevailing in the state. The concept and use of opportunity values is fundamental to the optimal allocation of scarce resources. Energy investment decisions should not be made solely on the basis of accounting prices in the state if the international value of traded energy commodities such as coal and gas diverge from local market prices.

The choice of a time horizon is also crucial. If a too short-term planning period is selected, the investment rankings and choices will differ markedly from those obtained through a more appropriate long-term perspective. In other words, the benefit-cost analysis would point to different generation expansion plans depending on the selected planning period. A short-run optimization of state incomes would, at best, allow only a moderate growth in fixed capital formation, at worst, it would lead to underinvestment in not only the energy sector but also in other infrastructure facilities such as roads, airports, hospitals, schools, and communications.

It therefore follows that the Susitna project, as other Alaskan investments, should be appraised on the basis of long-run optimization, where the long-run is defined as the expected economic life of the facility. For hydroelectric projects, this service life is typically 50 years or more. The costs of a Susitna-inclusive generation plan have therefore been compared with the costs of the next-best alternative which is the all-thermal generation plan and assessed over a planning period extending from 1982 to 2040, using internally consistent sets of economic scenarios and appropriate opportunity values of Alaskan energy.

Throughout the analysis, all costs and prices are expressed in real (inflation-adjusted) terms using January 1982 dollars. Hence, the results of the economic calculations are not sensitive to modified assumptions concerning the rates of general price inflation. In contrast, the financial and market analyses, conducted in nominal (inflation-inclusive) terms, will be influenced by the rate of general price inflation from 1982 to 2040.

(ii) Price Inflation and Discount Rates

- General Price Inflation

Despite the fact that price levels are generally higher in Alaska than in the Lower 48, there is little difference in the comparative rates of price changes; i.e. price inflation. Between 1970 and 1978, for example, the U.S. and Anchorage consumer price indexes rose at annual rates of 6.9 and 7.1 percent, respectively. From 1977 to 1978, the differential was even smaller: the consumer prices increased by 8.8 percent and 8.7 percent in the U.S. and Anchorage (1).

Forecasts of Alaskan prices extend only to 1986 (2). These indicate an average rate of increase of 8.7 percent from 1980 to 1986. For the longer period between 1986 and 2010, it is assumed that Alaskan prices will escalate at the overall U.S. rate, or at 5 to 7 percent compounded annually. The average annual rate of price inflation is therefore about 7 percent between 1982 and 2010. As this is consistent with long-term forecasts of the CPI advanced by leading economic consulting organizations, 7 percent has been adopted as the study value (3,4).

- Discount Rates

Discount rates are required to compare and aggregate cash flows occurring in different time periods of the planning horizon. In essence, the discount rate is a weighting factor reflecting that a dollar received tomorrow is worth less than a dollar received today. This holds even in an inflation-free economy as long as the productivity of capital is positive. In other words, the value of a dollar received in the future must be deflated to reflect its earning power foregone by not receiving it today. The use of discount rates extends to both real dollar (economic) and escalated dollar (financial) evaluations, with corresponding inflation-adjusted (real) and inflation-inclusive (nominal) values.

- Real Discount and Interest Rates

Several approaches have been suggested for estimating the real discount rate applicable to public projects (or to private projects from the public perspective). Three common alternatives include:

- the social opportunity cost (SOC) rate,
- the social time preference (STP) rate, and
- the government's real borrowing rate or the real cost of debt capital (5,6,7).

The SOC rate measures the real social return (before taxes and subsidies) that capital funds could earn in alternative investments. If, for example, the marginal capital investment in Alaska has an estimated social yield of X percent, the Susitna Hydroelectric Project should be appraised using the X percent measure of "foregone returns" or opportunity costs. A shortcoming of this concept is the difficulty inherent in determining the nature and yields of the foregone investments.

The STP rate measures society's preferences for allocating resources between investment and consumption. This approach is also fraught with practical measurement difficulties since a wide range of STP rates may be inferred from market interest rates and socially desirable rates of investment.

A sub-set of STP rates used in project evaluations is the owner's real cost of borrowing; that is, the real cost of debt capital. This industrial or government borrowing rate may be readily measured and provides a starting point for determining project-specific discount rates. For example, long-term industrial bond rates have averaged about 2 to 3 percent in the U.S. in real (inflation-adjusted) terms (3,8). Forecasts of real interest rates show average values of about 3 percent and 2 percent in the periods of 1985 to 1990 and 1990 to 2000, respectively. The U.S. Nuclear Regulatory Commission has also analyzed the choice of discount rates for investment appraisal in the electric utility industry and has recommended a 3 percent real rate (24). Therefore, a real rate of 3.0 percent has been adopted as the base case discount and interest rate for the period 1982 to 2040.

. Nominal Discount and Interest Rates

The nominal discount and interest rates are derived from the real values and the anticipated rate of general price inflation. Given a 3-percent real discount rate and a 7-percent rate of price inflation, the nominal discount rate is determined as 10.2 percent or about 10 percent*.

(iii) Oil and Gas Prices

- Oil Prices

In the base period (January 1982), the Alaskan 1982 dollar price of No. 2 fuel oil is estimated at $6.50/MMBtu.

Long-term trends in oil prices will be influenced by events that are economic, political and technological in nature, and are therefore estimated within a probabilistic framework.

As shown in Table 18.1, the base case (most likely escalation rate) is estimated to be 2 percent (to 2000) and 1 percent from 2000 to 2040. To be consistent with Battelle forecasts, a 2-percent rate was used throughout the OGP planning period 1982 to 2010 and 0 percent thereafter. In the low and high scenarios the growth rates were estimated at 0 percent (1982-2051), and 4 percent (to 2000), and 2 percent (beyond 2000), respectively. These projections are also consistent with those recently advanced by such organizations as DRI (9), World Bank (10), U.S. DOE (11), Canadian National Energy Board (12).

- Gas Prices

Alaskan gas prices have been forecast using both export opportunity values (netting back CIF prices from Japan to Cook Inlet) and domestic market prices as likely to be faced in the future by Alaskan electric utilities. The OGP analysis used market prices as estimated by Battelle, since there are indications that Cook Inlet reserves may remain insufficient to serve new export markets.

. Domestic Market Prices

Table 18.2 depicts the low, medium and high domestic market prices used in the OGP analysis. In the medium (most likely) case, prices escalate at real rates of 2.5 percent (1982 to 2000) and 2 percent (beyond 2000). In the low case, there is zero escalation and in the high case, gas prices grow at 4 percent (to 2000) and 2 percent (beyond 2000).

* (1 + the nominal rate) = (1 + the real rate) x (1 + the inflation rate) = 1.03 x 1.07, or 1.102

. Export Opportunity Values

Table 18.2 also shows the current and projected opportunity value of Cook Inlet gas in a scenario where the Japanese export market for LNG continues to be the alternative to domestic demand. From a base period plant gate price of $4.69 MMBtu (CIF Japan), low, medium and high price escalation rates have been estimated for the intervals 1982 to 2000 and 2000 to 2040. The cost of liquefaction and shipping (assumed to be constant in real terms) was subtracted from the escalated CIF prices to derive the Cook Inlet plant-gate prices and their growth rates. These Alaskan opportunity values are projected to escalate at 2.7 percent and 1.2 percent in the medium (most likely) case. Note that the export opportunity values consistently exceed the domestic prices. In the year 2000, for example, the opportunity value is nearly double the domestic price estimated by Battelle.

(iv) Coal Prices

The shadow price or opportunity value of Beluga and Healy coal is the delivered price in alternative markets less the cost of transportation to those markets. The most likely alternative demand for thermal coal is the East Asian market, principally Japan, South Korea, and Taiwan. The development of 60-year forecasts of coal prices in these markets is conditional on the procurement policies of the importing nations. These factors, in turn, are influenced to a large extent by the price movements of crude oil.

- Historical Trends

Examination of historical coal price trends reveals that FOB and CIF prices have escalated at annual real rates of 1.5 percent to 6.3 percent as shown below:

. Coal prices (bituminous, export unit value, FOB U.S. ports) grew at real annual rates of 1.5 percent (1950 to 1979) and 2.8 percent (1972 to 1979) (11).

. In Alaska, the price of thermal coal sold to the GVEA utility advanced at real rates of 2.2 percent (1965 to 1978) and 2.3 percent (1970 to 1978).

. In Japan, the average CIF prices of steam coal experienced real escalation rates of 6.3 percent per year in the period 1977 to 1981 (20,21). This represents an increase in the average price from approximately $35.22 per metric ton (mt) in 1977 to about $76.63/mt in 1981.

As shown below, export prices of coal are highly correlated with oil prices, and an analysis of production costs has not predicted accurately the level of coal prices. Even if the production cost forecast itself is accurate, it will establish a minimum coal price, rather than the market clearing price set by both supply and demand conditions.

. In real terms export prices of U.S. coal showed a 94-percent and 92-percent correlation with oil prices (1950 to 1979 and 1972 to 1979).*

. Supply function (production cost) analysis, has estimated Canadian coal at a price of $23.70 (1980 US$/ton) for S.E. British Columbia (B.C.) coking coal, FOB Roberts Bank, B.C., Canada (18,23). In fact, Kaiser Resources (now B.C. Coal Ltd.) has signed agreements with Japan at an FOB Price of about $47.50 (1980 US$/ton (19). This is 100 percent more than the price estimate based on production costs.

. The same comparison for Canadian B.C. thermal coal indicates that the expected price of $55.00 (1981 Canadian $) per metric ton (2,200 pounds) or about $37.00 (1980 U.S. $) per ton would be 60 percent above estimates founded on production costs (18, 19,23).

. In longer-term coal export contracts, there has been provision for reviewing the base price (regardless of escalation clauses) if significant developments occur in pricing or markets. That is, prices may respond to market conditions even before the expiration of the contract.**

. Energy-importing nations in Asia, especially Japan, have a stated policy of diversified procurement for their coal supplies. They will not buy only from the lowest-cost supplier (as would be the case in a perfectly competitive model of coal trade) but instead will pay a risk premium to ensure security of supply (see Battelle 18,23).

- Survey of Forecasts

Data Resources Inc. is projecting an average annual real growth rate of 2.6 percent for U.S. coal prices in the period 1981 to 2000 (9). The World Bank has forecast that the real price of steam coal would advance at approximately the same rate as oil prices (3 percent/a) in the period 1980 to 1990 (10). Canadian Resourcecon Ltd. has recently forecast growth rates of 2 percent to 4 percent (1980 to 2010) for subbituminous and bituminous steam coal (22).

* Analysis is based on data from the World Bank.

** This clause forms part of the recently concluded agreement between Denison Mines and Teck Corporation and Japanese steel makers.

- Opportunity Value of Alaskan Coal

. Delivered Prices, CIF Japan

Based on these considerations, the shadow price of coal (CIF price in Japan) was forecast using conditional probabilities given low, medium and high oil price scenarios. Table 18.3 depicts the estimated coal price growth rates and their associated probabilities, given the three sets of oil prices. Combining these probabilities with those attached to the oil price cases yields the following coal price scenarios, CIF Japan.

Scenario	Probability	Real Price Growth
Medium (most likely)	49 percent	2 percent (1982-2000) 1 percent (2000-2040)
Low	24 percent	0 percent (1982-2040)
High	27 percent	4 percent (1982-2000) 2 percent (2000-2040)

The 1982 base period price was initially estimated using the data from the Battelle Beluga Market Study (18). Based on this study, a sample of 1980 spot prices (averaging $1.66/MMBtu) was escalated to January 1982 to provide a starting value of $1.95/MMBtu in January 1982 dollars.*

As more recent and more complete coal import price statistics became available, this extrapolation of the 19 sample was found to give a significant underestimate of actual CIF prices. By late 1981, Japan's average import price of steam coal reached $2.96/ MMBtu.** An important sensitivity case was therefore developed reflecting these updated actual CIF prices. The updated base period value of $2.96 was reduced by 10 percent to $2.66 to recognize the price discount dictated by quality differentials between Alaska coal and other sources of Japanese coal imports, as estimated by Battelle (18).

* The escalation factor was 1.03 x 1.14, where 3 percent is the forecast real growth in prices (mid-1980 to January, 1982) at an annual rate of 2 percent, and 14 percent is the 18-month increase if the CPI is used to convert from mid-1980 dollars to January, 1982 dollars.

** As reported by Coal Week International in October, 1981, the average CIF value of steam coal was $75.50 per metric ton. At an average heat value of 11,500 Btu/lb, this is equivalent to $2.96/MMBtu.

. Opportunity Values in Alaska

.. Base Case - Battelle-Based CIF Prices, No Export Potential for Healy Coal

Transportation costs of $0.52/MMBtu were subtracted from the initially estimated CIF price of $1.95 to determine the opportunity value of Beluga coal at Anchorage. In January 1982 dollars, this base period net-back price is therefore $1.43. In subsequent years, the opportunity value is derived as the difference between the escalated CIF price and the transport cost (estimated to be constant in real terms). The real growth rate in these FOB prices is determined residually from the forecast opportunity values. In the medium (most likely) case, the Beluga opportunity values escalate at annual rates of 2.6 percent and 1.2 percent during the intervals 1982 to 2000 and 2000 to 2040, respectively.

For Healy coal, it was estimated that the base period price of $1.75/MMBtu (at Healy) would also escalate at 2.6 percent (to 2000) and 1.2 percent (2000 to 2040). Adding the escalated cost of transportation from Healy to Nenana results in a January 1982 price of $1.75/MMBtu.* In subsequent years. the cost of transportation of which 30 percent is represented by fuel cost (which escalates at 2 percent) is added to the Healy price resulting in Nenana prices that grow at real rates of 2.3 percent (1982 to 2000) and 1.1 percent (2000 to 2040). Table 18.3 summarizes the real escalation rates applicable to Nenana and Beluga coal in the low, medium and high price scenarios.

.. Sensitivity Case - Updated CIF Prices, Export Potential for Healy Coal

The updated CIF price of steam coal (2.66/MMBtu after adjusting for quality differentials) was reduced by shipping costs from Healy and Beluga to Japan to yield Alaskan opportunity values. In January 1982, prices are $2.08 and $1.74 at Anchorage and Nenana, respectively. The differences between escalated CIF prices and shipping costs result in FOB prices that have real growth rates of 2.5 percent and 1.2 percent for Beluga coal and 2.7 percent and 1.2 percent for Healy coal (at Nenana). Table 18.3 shows escalation rates for the opportunity value of Alaskan coal in the low, medium and high price scenarios, using updated base period values.

* Transportation costs are based on Battelle (18,23).

(v) Generation Planning Analysis - Base Case Study Values

Based on the considerations presented in Sections (i) through (iv) above, a consistent set of fuel prices assembled for the base case probabilistic generation planning (PGP) analysis, as shown in Table 18.4. The study values include probabilities for the low, medium and high fuel price scenarios. The probabilities are common for the three fuels (oil, gas and coal) within each scenario in order to keep the number of generation planning runs to manageable size. In the case of the natural gas prices, domestic market prices were selected for the base case analysis with the export opportunity values used in sensitivity runs. The base period value of $3 was derived by deflating the 1996 Battelle prices to 1982 by 2.5 percent per year. Coal prices were also selected from the base case using Battelle's 1980 sample of prices as the starting point, with the updated CIF prices of coal reserved for sensitivity runs. Oil prices have been escalated by 2 percent (1982 to 2040).

(b) Analysis of Net Economic Benefits

(i) Modeling Approach

Given the economic parameters discussed in the previous section, the alternatives for electrical energy generation in the Railbelt were analyzed by comparing the production costs of electricity with and without the Susitna project. The primary tool for the benefit cost analysis was a generation planning model (OGP) which simulates production costs over a planning period extending from 1982 to 2051.

The method of comparing the "with" and "without" Susitna scenarios is based on total system costs. The planning model determines the total production costs of alternative plans on a year-by-year basis. These total costs for the period of modeling include all costs of fuel and operation and maintenance (O&M) for all generating units included as part of the system, as well as the annualized investment costs of any production plants added during the period of study. Factors which contribute to the ultimate consumer cost of power which are not included in this model are: all investment cost for plants in service prior to 1993, costs of the transmission and distribution facilities already in service, and administrative costs of utilities. These costs are common to all scenarios and therefore have been omitted from the study, as they have no differential impact on alternative generation plans.

In order to aggregate and compare costs, all annual costs from the 1993 to 2051 production simulations have been converted to a 1982 present worth (PW). These PWs are computed as the sum of two components. The first is the 1982 PW of the first 18 years of model study from 1993 to 2010. The second component is the estimated PW of long-term system costs from 2011 to 2051.

To model the system for an additional 40 years would require the development of further load forecasts and generation alternatives that are beyond the realm of any prudent projections. For this reason, the final study year (2010) production costs were assumed to simply recur for an additional 41 years, and added to the 18-year PW, to establish the long-term cost differences between alternative methods of power generation.

(ii) Base Case Analysis

- Pattern of Investments With and Without Susitna

The base case comparison of the with and without Susitna plans is based on production cost simulation for the period 1993 to 2051, using mid-range values for the load forecast, base period fuel prices, fuel price escalation, base period capital costs and capital cost escalation. Load forecasts, fuel prices and construction costs are analyzed in Chapters 5, 18.1(b) and 16, respectively. As discussed in Section 18.1(b), a real interest and discount rate of 3 percent is used.

The with-Susitna plan calls for 680 MW of capacity at Watana to be available to the system in 1993. Although the project can come on-line in stages during that year, for modeling purposes full-load generating capability is assumed to be available for the whole year. The second stage of Susitna, the Devil Canyon project, is scheduled to come on-line in 2002. The running of the project was tested for earlier and later dates and selection of the year 2002 found to result in the lowest long-term cost. Devil Canyon will have 600 MW of installed capacity.

The without-Susitna plan is discussed in Section 6.7. It includes 3,200-MW coal-fired plants added in Beluga in 1993, 1994, and 2007. A 200-MW unit at Nenana is added in 1996. In addition, 970-MW gas-fired gas turbines (GTs) are added during the 1997 to 2009 period.

- Base Case Net Economic Benefits

The economic comparison of these plans is shown in Table 18.5.

During the 1993 to 2010 study period, the 1982 PW cost for the Susitna plan is $3,119 million. The annual production cost is $385.3 million in 2010. The present worth of this level cost for a period extending to the end of the life of the Devil Canyon project (2051) is $3,943 million. The resulting total cost of the Susitna-inclusive plan is $7.06 billion in real (1982) dollars, presently valued to 1982.

The non-Susitna plan modelled has a 1982-PW cost of $3,213 million for the 1993 to 2010 period. With a 2010 annual cost of $491 million, the total long-term cost has a PW of $8.24 billion. Therefore, the net economic benefit of adopting the Susitna plan is $1.18 billion.

In other words, the present valued cost difference between the Susitna plan and the next-best thermal expansion plan is $1.2 billion in 1982 dollars. The 1982 present valued cost advantage of the Susitna plan ($1.2 billion) is equivalent to a 1982 per capita net economic benefit of $2,700 in Alaska. Expressed in 1993 dollars (the on-line date of Watana), this cost saving would have a levelized value of $2.5 billion.* It is noted that the magnitude of net economic benefits ($1.2 billion) is not sensitive to alternative assumptions concerning the overall rate of price inflation as measured by the CPI. The analysis has been carried out in real (inflation-adjusted) terms and therefore the present valued cost savings will remain at $1.2 billion regardless of CPI movements, as long as the real (inflation-adjusted) discount and interest rates are maintained at 3 percent.

The Susitna project's internal rate of return (IRR) has also been determined. This is the real (inflation-adjusted) discount rate at which the with-Susitna plan has zero net economic benefits, that is the discount rate at which the costs of the with Susitna and the "alternate" plans have equal costs. This IRR is about 4.1 percent in real terms, and 11.4 percent in nominal (inflation-inclusive) terms. Therefore, the Susitna investments would significantly exceed the 5 percent nominal rate of return test proposed by the State of Alaska in cases where state appropriations may be involved.**

It is emphasized that these net economic benefits and the rate of return stemming from the Susitna project are inherently conservative estimates due to several assumptions used in the OGP analysis.

. <u>Zero Growth in Far-Term Costs</u>

From 2010 to 2051, the OGP analysis assumed constant annual production costs in the Susitna and the non-Susitna plans. This has the effect of excluding real escalation in fuel prices and the (replacement) capital costs of thermal plants, and thereby underestimating the long-term costs of thermal generation plants.

. <u>Loss of Load Probabilities</u>

The loss of load probability in the non-Susitna plan is calculated at 0.099. This means that the system in 2010 is on the verge of adding an additional plant, and would do so in 2011. These costs are however, not included in the anlaysis which is cut off at 2011. On the other hand, the Susitna plan has a loss of load probability of 0.025, and may not require additional capacity for several years beyond 2010.

*$1.2 billion x 2.105, where 2.105 is the general price inflation index for the period 1982 to 1993.

**See State of Alaska's SB 25, Section 44.83.470.

. Far-Term Energy From Susitna

Some of the Susitna energy output (about 344 GWh) is still not used by 2010. This energy output would be available to meet future increases in projected demand in the summer months. No benefit is attributed to this energy in the analysis.

. Equal Environmental Costs

The OGP analysis has implicitly assumed equal environmental costs for both the Susitna and the non-Susitna plans. To the extent that the thermal generation expansion plan is expected to carry greater environmental costs than the Susitna plan, the economic cost savings from the Susitna project are understated. It is conceivable that these so-called negative externalaties from coal-fired electricity generation will have been mitigated by 1993 and beyond, from the enactment of new environmental legislation. However, such government action action would simply internalize the externality by forcing up the production market costs of thermal power.

(iii) Sensitivity Analysis

A sensitivity analysis has been carried out to identify the impact of modified assumptions on net benefits. The analysis was directed at the following variables:

- load forecast
- real interest and discount rate
- construction period
- period of analysis
- capital costs
  . Susitna
  . Alternatives
- O&M costs
- base period coal price
- real escalation in capital and O&M costs and fuel prices
- system reliability
- Chackachamna included in non-Susitna plan.

Tables 18.6 to 18.13 depict the results of the sensitivity analysis. In particular, Table 18.13 summarizes the net economic benefits of the Susitna project associated with each sensitivity test. The net benefits have been compared using indexes relative to the base case value ($1.2 billion) which is set to 100.

The greatest variability in results occurs in sensitivity tests pertaining to fuel escalation rates, discount rates, and base period coal prices. For example, a scenario with high fuel price escalation results in net benefits that have a value of 253 relative to the base case. In other words, the high case provides 253 percent of the base case net benefits. In general, the Susitna plan maintains its positive net benefits over a reasonably wide range of values assigned to the key variables.

A multi-variate analysis in the form of probability trees is also currently being undertaken to test the joint effects of varying several assumptions in combination rather than individually. This probabilistic analysis will provide a range of expected net economic benefits and probability distributions that identify the chances of exceeding particular values of net benefits at given levels of confidence.

18.2 - Risk Analysis

A risk analysis was undertaken to provide a basis for determining the extent to which perceived risks are likely to influence capital costs and schedule. In addition, because a mature Susitna project would represent a major portion of the total generation system, a further risk analysis was made to assess the probability and consequences of a long-term outage of the proposed transmission system. This section summarizes both risk analyses. A more detailed report is included in the project documentation for Subtask 11.03, Risk Analysis.

(a) Approach

Any major construction effort is inevitably exposed to a large number of risks. Low probability magnitude floods may occur at critical periods of construction: accidents may happen: sub-surface investigations, no matter how thorough, cannot always predict actual conditions uncovered when the major excavations are undertaken: the normal estimating process implicity assumes a set of reasonably "normal" expectations as direct costs are developed, adding a contingency to the directly computed total on the grounds that problems usually do occur even though their specific nature may not be accurately foreseen at the outset.

The Susitna risk analysis took explicit account of 21 different risks, applying them, as appropriate, to each major construction activity. The effort involved combining reasonably precise data (e.g. the probability that a particular flood crest will occur in any given year can be determined from analysis of hydrologic records) with numerous subjective judgements (e.g. until a particular flood crest does occur, it cannot be known with any degree of certainty what damage it will cause. The overall methodology is illustrated in Figure 18.4-1.

(b) Elements of the Analysis

Figure 18.4-2 graphically depicts important questions which were addressed at the start and relates them to elements of the analysis. Each element is further subdivided as follows:

(i) Configurations

Three primary configurations were considered:

- The Watana hydroelectric project (with transmission);
- The Devil Canyon hydroelectric project (with transmission); and
- The Susitna transmission system alone.

Separate risk studies of these configurations permitted the production of data which can be aggregated in various ways to accommodate alternative "power-on-line" dates which differ according to the various demand forecasts.

(ii) Congiguration States

Two configuration states were considered:

- Construction Period - applicable to Watana and Devil Canyon

- Operation Period - applied only to the transmission system configuration.

(iii) Risks

Twenty-one risks were identified for consideration in the analysis and were grouped as follows:

- Natural Risks

  . flood
  . ice
  wind
  seismic
  permafrost deterioration
  . geologic conditions
  . low streamflow

- Design Controlled Risks

  . seepage piping erosion
  . ground water

- Construction Risks

  . equipment availability
  . labor availability strikes
  . material availability
  . equipment breakdown
  . matedrial deliveries
  . weather

- Human Risks

  . contractor capability
  . construction quality control
  . accidents
  . sabotage vandalism

- Special Risks

  . regulatory delay
  . estimating variance

(iv) Activities

For each configuration state involving construction, up to 22 activities were considered. For Watana, for example, these included:

- main access
- site facilities
- diversion tunnels
- cofferdams
- main dam excavation
- main dam fill initial portion
- main dam fill final portion
- relict channel protection
- chute spillway
- emergency spillway
- service spillway tunnels
- intake
- penstock
- powerhouse
- transformer gallery
- tailrace and surge chambers
- turbine-generators
- mechanical electrical equipment
- switchyard
- transmission
- impoundment
- test and commission.

(v) Damage Scenarios

Up to ten different damage scenarios were associated with each logical risk-activity combination. While these varied significantly from one risk-activity combination to another, they generally described a range of possibilities which accounted for discrete increments extending from "no damage" to "catastrophic loss".

(vi) Criteria

The consequences of realizing particular risk magnitudes for each activity were measured in terms of the following criteria:

- cost implications
- schedule implications
- manpower requirements

(vii) Boundary Conditions

The following assumptions and limitations were established to permit a reasonable and consistent analysis of the problem:

- All cost estimates were made in terms of January 1982 dollars. Thus, results are presented in this report in terms only of real potential cost variations, exclusive of inflation.

- The analysis was limited only to the construction periods for Watana and Devil Canyon since the greatest potential cost and schedule variance would be possible during these periods. The risk analysis for the operating period was associated solely with the transmission system since that configuration represents the most likely source of a major system outage during the project operation.

- The risk analysis was accomplished concurrently with finalization tion of the total project cost estimate and was necessarily associated with the feasibility level design. There is clearly some potential for design change as the project proceeds; a further risk analysis should be undertaken coincident with completion of final detailed design and prior to committment to major construction activities. Even so, the "estimating variance" risk takes into account the fact that some design changes are likely to appear as detailed design effort proceeds.

- A great deal of subjective judgement was necessarily involved in assessing certain probabilities and in predicting possible damage scenarios. This effort was accomplished initially by individual qualified professionals in the various disciplines and was subjected to iterative group review and feedback efforts. To the extent that individual biases entered the analysis, their effects were probably mutually offsetting. Even so, sensitivity tests were made for risks which were important contributors to the final results.

- The risk list does not include the important possibility of funding delays or of financing problems. These issues were dealt with in a separate financial risk analysis as discussed in paragraph 18.5 below.

(c) Risk Assessments

For each of the risks identified in paragraph 18.2(b) (iii) above, the assessment commenced with detailed definition of credible events. Where flood was identified as a risk, for example, the potential magnitudes and associated probabilities of the floods was estimated. Data sources ranged from reasonably accurate scientific data (particularly applicable to the natural risk category), historical experience on water resources projects, to subjective group judgements where data gaps existed.

In each case, the maximum credible event was first established. This choice set an upper limit on a scale of possible events starting at "no damage" situation. Continuing with flood as an example, the maximum credible event was considered to be the probable maximum flood which had been computed in the hydrologic studies (corresponding to a return period of more than 10,000 years and an annual probability of occurrence of less than .0001).

Once risks were defined and logical risk-activity combinations were reviewed, the consequences of realizing each selected risk magnitude were considered (if this risk magnitude is realized, will a partially completed structure be damaged? Will it fail? If it fails, is some other work in progress disrupted?). Because of the uncertainties associated with these projections, a range of damage scenarios and associated probabilities of them occuring was established.

Even if a particular risk level is realized and a particular damage is suffered, the cost and schedule of restoring the activity are difficult to precisely establish. Each of the risk analysts therefore provided three values for each criterion:

- a minimum value corresponding to the one time in twenty that the weather is particularly good, materials are readily available, no accidents occur, etc.

- a modal value associated with the most likely expectation of the analyst;

- a maximum value corresponding to the one time in twenty that everything is more difficult than expected.

In the computerized calculation process, the three criterion values supplied by the risk analyst were fitted to a triangular distribution, which approximated the beta distribution illustrated at the bottom of Figure 18.2-3.

In effect, then, designation of the three conceptual criterion values led to generation of a histogram with relatively narrow intervals and a nearly continuous range of possible values over a relatively wide spectrum.

Figure 18.2-3 illustrates the structural relationship for handling risk-activity combinations, damage scenarios, and criterion values.

While the procedure described above is generally applicable, some commentary on particular aspects of its application and on certain unique risks is appropriate:

(i) The terminology "damage scenario" has been used for convenience since most identified risks will normally be thought of as reasons that the cost will be higher than had been estimated or that the schedule will be exceeded. In fact, however, the process does permit consideration of what might be regarded as a "negative" damage scenario. The geologic conditions risk is an excellent example. The cost estimate was produced on the basis of estimates of requirememnts for some concrete lining in the penstocks, extensive grouting, a certain level of rock bolting, and the like. If geologic conditions are found to be better than currently assumed, the costs could be less and the schedule might be accelerated.

(ii) The estimating variance risk was treated in a special way because it cannot easily be conceptualized in physical terms. It accounts for inevitable differences which do occur between estimates and actual bids, and between bids and actual activity costs - even in the absence of any other identified risks. Its probability of occurrence and associated range (fractions or multiples of the basic estimate) were determined from historical data on water resources projects. It includes, but is not necessarily limited to, such considerations as:

- the influence of competition and market pressures
- estimating discrepancies or errors in unit quantities on the parts of both owner's estimator and bidder
- particular contract forms and the owner's acceptance non-acceptance of certain risks;
- labor market conditions and the nature of project labor agreements
- productivity and efficiency changes over time;
- the cost implications of variances between activity schedules and actual activity durations;
- the potential for scope changes over time;
- extraordinary escalation of construction costs above the underlying inflation rate.

(iii) In addition to estimating variance, a second special risk is associated with regulatory matters. Various legislated controls will most certainly be applied to the Susitna project and it is a a relatively simple matter to compute the minimum time in which regulatory requirements could be satisfied. It is a far more difficult task indeed to estimate the precise nature and duration of possible future regulatory delays. It would also clearly be inappropriate to attempt to apply regulatory risks at the activity level.

This risk was handled by developing a separate distribution for a range of periods necessary for satisfaction of important licensing and permitting requirements.

Data used in arriving at a distribution were based on recent experiences on other water resources projects as well as on discussions with staff members of the Federal Energy Regulatory Commission. The effect of applying the regulatory risk is primarily one of shifting the starting time for commencement of construction activities, leading to corresponding change in the projected completion time. A lesser effect of the regulatory risk was to introduce delays during construction.

Regulatory requirements have been an important influence during the past decade on major construction costs and schedules, though it is difficult to isolate their effects. In order to separately consider estimating variance risks and regulatory risks, "estimating variance" probability determination relied heavily upon water resources construction data developed for projects essentially completed prior to the passage of the National Environmental Policy Act (NEPA). As noted above, regulatory risk probability distributions were derived from more recent projects.

(iv) Each of the various risk magnitude probabilities was originally calculated as an annual value. On a risk-activity by risk-activity basis, these annual values were then converted by standard computational procedures to provide a probability of occurrence during the duration of the acitivty.

(v) The concept of "response is particularly important in the formal risk analysis process. As the terminology suggests, a "response" represents the action to be taken if a particular event occurs. There are two kinds of "response". The first - and most often used - is an expected reaction to the occurrence of a particular damage level (i.e. if this damage level is incurred, then what action must be taken to restore the activity to its pre-damage status? And what cost, schedule, and manpower implications (consequences) will result?). A second kind of response can also be considered and it provides an important link between the design team and the risk analysis team. This latter type is the "preventive response" (i.e. what changes might reasonably be made in the design and or construction procedures whichwould permit us to avoid or reduce a particular damage level? Is the cost and schedule change which might ensue worthwhile when compared to the probability and magnitude of the consequences which would otherwise be incurred?) A number of preventive responses were identified by risk analysts during the risk study and several of these were incorporated into the project design and dedsign criteria. There may be further opportunities for preventive response. Since none would be chosen unless it offered a net benefit to cost and or schedule, it may reasonably be concluded that as detailed design proceeds and as subsequent risk analysis updates areaccomplished, a gradual reduction in the spread of possible values can be expected.

(d) Interpretation of Results

(i) Presentation of Data

Minor variations in activity costs were generated by the estimating team concurrent with development of the risk analysis. In addition, account was taken of the expectation that construction costs will escalate at a faster rate than normal inflation - both in the economic analyses and the risk analyses. To avoid confusion regarding absolute cost values, the results of the risk analysis are presented in this section as percentages of the estimated project cost or as ratios between actual costs and estimated costs.

(ii) Watana Cost-Probability Distribution

Figure 18.2-5 illustrates the cumulative distribution of total direct costs and their related non-exceedance probabilities as determined in the risk analysis. Certain important points noted on the figure are interpreted as follows:

- The project direct cost estimate, including contingencies, was presented in Chapter 16. Point "A" on Figure 18.2-5 corresponds to this project estimate: the analysis suggests that the probability of completing Watana for less than the project estimate which includes a 17.5 percent contingency allowance.

- Point "B" corresponds to the "low" cost estimate which was tested for sensitivity in the OGPS system cost analysis. The probability that Watana will be completed for less than this cost estimate is about 46 percent.

- Point "C" on Figure 18.2-5 corresponds to a cost equal to the "high" estimate tested in the OGPS analysis to determine the effect of such a cost on total project economics. The risk analysis suggests that there is a 90 percent probability that this cost will not be exceeded.

- As will be noted from Figure 18.2-5, there remains a small but measurable possibility that the project costs will exceed even the "high" estimate value at Point "C". It can be argued that the degree of conservatism which was used in the analysis has overstated the possibility of extreme upper limits on total cost. Paragraph (v) below addresses this issue, comparing these results with historical data.

- The expected value of the actual cost is 90.25 percent of the project estimate.

(iii) Devil Canyon - Probability Distributions

Figure 18.2-6 provides the cumulative probability distribution for Devil Canyon costs. Points A, B, and C on the curve correspond to those discussed above for Watana and are associated with probabilities of 74 percent, 47 percent, and 90 percent, respectively, for actual percentages of the project estimate being less than indicated values. Once again, a not insignificant long "tail" in the extreme upper righthand portion of the distribution provides a measure of the potential exposure to large overruns. The expected value of the actual cost is 91.5 percent of the project estimate.

(iv) Total Project Distribution

Figure 18.2-7 combines the separate Watana and Devil Canyon projects, providing a cumulative distribution for the Susitna Hydroelectric Project as a whole. Points A, B, and C now have associated probabilities of non-exceedance of 73 percent, 47 percent and

90 percent, respectively, suggesting that a broad range of total project cost ratios are possible. In the 10 percent range and 90 percent probability interval, the cost range spans nearly three billion dollars. If the project follows historical patterns, it may be expected that this range will narrow over time as detailed design and construction proceed. Note that the cost distributions are in every case based upon January 1982 do dollars and do not account for the effects of inflation. Interest during construction or finance charges are not included. Only the potential for extraordinary construction cost escalation (over and above inflation) has been taken into account. It follows that if the project is completed in the next several decades, the final "actual" cost will have to be adjusted to equivalent 1982 dollars if it is to be compared with risk analysis results presented herein.

(v) Comparison with Available Data

During the assessment of the important "estimating variance" risk (see paragraph 18.2(c) (ii) above), historical data for 49 federal water resources projects completed prior to passage of NEPA were considered. Figure 18.2-8 offers a cumulative probability historical program for various cost ratios. In each case, the cost ratio reflects the actual project cost (after adjustment for inflation) divided by the "initial" estimated cost. It will be seen that relatively large overruns have occurred in the past, while there is also evidence that a substantial number of water resources projects have been accomplished for less than the originally estimated costs.

In order to compare this information with the Susitna Risk Analysis results, it is necessary to determine the meaning of "initial" estimate in terms of the historical data. In each case, the "initial" estimate is the estimate presented to the Congress at the time that a request was made for project authorization. Thus, it would be inappropriate to regard the current Susitna estimate (as discussed in Chapter 16) as an "initial" estimate in the federal sense. Fortunately, however, the Susitna project does have a long history of federal involvement. Indeed, the Corps of Engineers provided a detailed "initial" estimate in 1975 as the basis for seeking authorization for important design activities. This "initial" estimate was further updated by a second "initial" estimate in 1979 after some additional exploratory work and further analysis were requested by the Office of Management and Budget. Inclusive of contingencies and excluding lands, the direct cost "initial" Corps of Engineers' estimate (from the 1979 report) in January 1982 dollars for the Watana Devil Canyon (thin arch dam) project was used as the denominator for display of possible Susitna cost ratios.

Figure 18.2-9 overlays the results of the Susitna risk analysis on the historical data. Note that the cost ratios differ on this display from those on Figure 18.2-7 because of the necessity to use the "initial" estimate for comparison purposes.

As may be seen from Figure 18.2-9, the Susitna risk analysis results reflect a more pessimistic expectation at low cost levels than the historical data would appear to indicate is appropriate.

(vi) Schedule Risks

At the same time that minimum, modal, and maximum cost values were estimated for each damage scenario in each risk-activity set, estimates were also made of similar values for potential schedule changes. As a result, schedule probability distributions were generated for each major activity. However, these individual distributions could not be combined in the same manner in which the cost data were handled.

A critical path network was prepared for the entire set of activities for each configuration. Individual probability distributions for critical activities were then combined tc yield a distribution for the total project schedule.

Several critical paths were identified in theprocess, since a long delay on a non-cirtical activity can, of course, place that activity on a new critical path. The "raw" schedule delay distribution was then considered in the context of a one-year schedule contingency which had been suklt into the original estimate and in light of regulatory delay risks. The resulting distributions are discussed and interpreted as follows:

- Figure 18.2-12 provides a cumulative probability distribution in months from the scheduled completion data for the Watana project. It reflects all risk contributions except those posed by regulatory requirements. It is based upon a critical path through the main dam construction and takes into account the one-year schedule contingency. The indicated probability of completing the project ahead of schedule or on time is about 65 percent. There is only a 17-percent chance of completing the project a year early (i.e. in 1992).

- Figure 18.2-13 provides a similar distribution after regulatory risks are accounted for. Two components are included: (1) prior to the start of construction, a license must be issued by the Federal Energy Regulatory Commission. There is a small chance (estimated to be 25 percent) that the license will be issued a year earlier than the current 30-month licensing schedule anticipates. The probability of meeting or improving upon the 30-month estimate is about 72 percent and there is a 90-percent probability that not more than 38 months will be required; (2) during the construction period, regulatory delays may be imposed as a result of various permitting requirements, injunctions, etc. These delays yield only increases in schedule and range from a 50-percent probability of delays of a month or less to a 95 percent probability that regulatory delays during construction will not exceed 12 months.

As may be seen from Figure 18.2-13, the net effect of the regulatory risks is to broaden the range of possible values. At the lower end of the distribution, it will be noted that the chances of completing at least a year early will have increased to nearly 40 percent - primarily because of the chance of getting a license early and therefore, starting early. No significant change appears for the probability of meeting or improving upon the schedule. A substantial effect is evident in the upper portion of the curve where the chances of long regulatory delays have pushed out the 95 percent confidence level to an expectation of no more than three months attributable to risks other than regulatory, as may be seen on Figure 18.2-12.

While similar distributions cna be plotted for Devil Canyon, they are less meaningful since there is flexibility associated with its starting date.

(vii) Transmission Line Risks

The separate risk analysis of the Susitna transmission system was conducted to determine the probability of significant power supply interruptions at the two major load centers in Anchorage and Fairbanks. The methodology was generally similar to that described in preceding paragraphs. Recognizing that the system is assumed to be be in an operating mode, those risks which had applied only for construction in the preceding analysis (e.g. contractor capability) were eliminated from the risk list. Additions to the list were made to account for the potential effects of lighting, aircraft, collisions, and anchor-dragging in Knik Arm (applicable to the submarine cable segment). Account was taken of redundancies designed into the system (e.g. a loss of one line in the three-line system extending south toward Anchorage can be tolerated with no loss of energy delivery capability).

In addition, special attention was given to dependencies (e.g. an earthquake which causes the loss of two lines will very likely knock out the third. On the other hand, vandalism which causes an outage on one line is only infrequently expected to extend to all lines). Important assumptions included the availability of well-trained repair crews and equipment, and a reasonable supply of spare components.

The results of the analysis provide the cumulative probability of not exceeding a given number of days of reduced energy delivery capability. Figures 18.2-14 and 18.2-15 display this information for Anchorage and Fairbanks, respectively. Interpretations are as follows:

- In the particular case of Anchorage igure 18.2-14), it will first be noted that the probability scale includes only the extreme upper range of non-exceedance probabilities. The intersection of the distribution curves on the probability axis indicates that the probability of no lost energy delivery capability in a given year is 0.958 and of not having 50 percent reduction of 0.955. Beyond these points the curves rise sharply, indicating that outages beyond five days are extremely unlikely. The "expected" annual value of 0.06961 da days for a total delivery loss may be compared with the "loss of load probability" of 0.1 (one day in ten years) which had been used in the generation planning efforts in the economic studies. In short, the risk analysis confirms that the reliability of the transmission system for energy delivery to Anchorage is consistent with the requirements of the overall Railbelt generation system. The "expected" annual value of 0.09171 days for a 50-percent reduction in energy delivery a appears to be similarly acceptable when compared to assumed loss of load probability.

- The cumulative probability distribution for Fairbanks (Figure 18.2-15) has a slightly different intercept on the probability axis and its shape is also slightly different from those for Anchorage. These differences stem from the fact that delivery to Fairbanks requires no submerged crossing and certain other risks (e.g. flood, temperature extremes) would be expected to have different probabilities for northern and southern segments of the system. In spite of the absolute differences, it may be seen from the display that "expected" annual value of .08116 does not exceed the loss of load probability criterion of 0.1 day per year. No 50-percent loss for Fairbanks is shown since the loss of one of two lines causes no reduction in delivery capability. Two lines lost is, of course, a 100-percent loss.

(viii) Emergency Response

In spite of the apparent reliability of the transmission system, it is nonetheless true that a small but finite chance of relatively long-term outages does exist. It is also unfortunately true that certain extreme risk magnitudes (e.g. combination of extreme loss temperature, wind, and ice) which could lead to an outage also tend to coincide with high demands by users on the generating system. The "response" in this case is extremely important. The final report for Subtask 11.03, Risk Analysis, provides such a response in the form of a preliminary emergency plan which includes such measures as shedding non-essential loads, putting reserve capacity on line, and energy transfers from military generation systems. Prior to the time that the Susitna Hydroelectric Project begins operation, this plan should be updated and occasional tests should be made.

(e) Conclusions

Based upon the risk analysis, it is concluded that:

(i) The probabilities that actual costs will not exceed values subjected to sensitivity tests in the economic anlaysis are are as follows:

Value	Probability That Value Will Not Be Exceeded
Project Estimate	73 %
Low Capital Cost Tested in the Economic Analysis	47 %
High Capital Cost Tested in the Economic Analysis	90%

(ii) Exposure to potential costs above the project estimates does exist and there is about a one 1 percent chance that an overrun of 40 percent or more (in 1982 dollars) will occur.

(iii) The annual probability that no interruption in energy delivery to major load centers will occur as a result of transmission line failures is in excess of 95 percent.

Expected values of energy delivery interruptions are less than one day in ten years and are consistent with loss of load probabilities assumed in the generation planning efforts.

(iv) There is a 65-percent probability that the Watana rpoject will be completed prior to the scheduled time in 1993. Exposure to schedule delays is heavily influenced by regulatory requirements and there is a 10-percent probability that the Watana project will not be completed until 1995 or later.

18.3 - Marketing

This section presents an assessment of the market in the Railbelt Region for the energy and capacity of the Susitna development. A range of rates at which this power could be priced is presented together with a proposed basis for contracting for the supply of Susitna energy.

(a) The Railbelt Power System

Susitna capacity and energy will be delivered to the "Railbelt Region Interconnected System" which will result from the linkage of the Anchorage and Fairbanks systems by an intertie to be completed in the mid-1980's.

The Railbelt Region covers the Anchorage-Cook Inlet area, the Fairbanks-Tenana Valley area, and the Glennallen-Valdez area (Figure 18.14). The utilities, military installations and universities within this area which own electric generating facilities are listed in Table 18.14. The service area of these utilities is shown in Figure 18.15 and the generating plants serving the region are listed in Table 18.15.

The Railbelt Region is currently served by nine major utility systems; five are rural electric cooperatives, three are municipally owned and operated, and one is a federal wholesaler. The relative mix of electric generating technologies and types of fuel used by the Railbelt utilities in 1980 are summarized in Figure 18.16.

In 1980, the Anchorage-Cook Inlet area had 81 percent, the Fairbanks-Tenana Valley area 17 percent, and the Glennallen-Valdez area 2 percent of the total energy sales in the Railbelt Region.

If the recommendations of the May 1981 Gilbert/Commonwealth Report are implemented, the Anchorage and Fairbanks power systems will be intertied before the Susitna project comes into operation. The proposed intertie will allow a capacity transfer of up to 70 MW in either direction. The proposed plan of interconnection envisages initial operation at 138 kV with subsequent uprating to 345 kV allowing the line to be integrated into the Susitna transmission facilities.

(b) Regional Electric Power Demand and Supply

A review of the socioeconomic scenarios upon which forecasts of electric power demand were based is presented in Section 5 of this report. The forecasts adopted here are the mid-range levels presented by Battelle Northwest in December 1981. Subsequent forecasts which introduce price demand considerations have not been considered at this stage. The results of studies presented in Section 5 call for Watana to come into operation in 1993 and to deliver a full year's energy generation in 1994. Devil Canyon will come into operation in 2002 and deliver a full year's energy in 2003. Energy demand in the Railbelt Region and the deliveries from Susitna are shown in Figure 18.17.

(c) Market and Price for Watana Output in 1994

It has been assumed 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 a low cost alternative source of energy. On this basis it is estimated that for the 3315 GWh of energy generated by Watana in 1994 to be attractive, a price of 140 mils per kWh in 1994 dollars is required. Justification for this price is illustrated in Figure 18.18. Note that the assumption is made that the only capital costs which would be avoided in the early 90s would be due to new coal-fired generating plants (i.e. the 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 mils kWh price are considered in Section 18.4 of this report; 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 mils, 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.

(d) Market Price for Watana Output 1995-2001

After its initial entry into the market in 1994, the price and market for the 3387 MWh 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: these savings per unit of output are illustrated in Figure 18.19.

(e) 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 18.20. Dividing these total savings by the energy contributed by Susitna indicates a price of 250 mils per kWh would be 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, as by this time period these thermal power facilities will be approaching retirement.

Only about 85 percent of the total Susitna output will be absorbed by the system in 2002, the balance of the output being progressively absorbed over the following decade. This will provide increasing total savings to the system from Susitna, with no associated increase in costs.

(f) 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 even when the output is priced up to the cost of the best thermal alternative.

18.4 - Financial Evaluation

(a) Forecast Financial Parameters

The financial, economic, and engineering estimates used in the financial analysis are summarized in Table 18.16. The interest rates and forecast rates of inflation (in the CPR) are of especial importance. They have been based on the forecast inflation rates in CPR and forecast interest rates on industrial bonds as given by Data Resources Incorporated, and conform to a range of other authoritative forecasts. To allow for the factors which have brought about a narrowing of the differential between tax-exempt and non-tax-exempt securities, it has been assumed that any tax-exempt financing would be at a rate of 80 percent rather than the historical 75 percent or so of the non-exempt interest rate. This identifies the forecast interest rates in the financing periods from 1985 in successive five year periods as 8.6 percent, 7.8 percent, and 7.4 percent. The accompanying rate of inflation is around 7 percent. In view of the uncertainty attaching to such forecasts and in the interest of conservatism, the following financial projections have been based upon the assumption of a 10-percent rate interest for tax-exempt bonds and an ongoing inflation rate of 7 percent.

(b) The Inflationary Financing Deficit

The basic financing problem of Susitna is the magnitude of its "inflationary deficit." Under inflationary conditions these deficits (early year losses) are an inherent characteristic of almost all debt financed long life, capital intensive projects (see Figure 18.21). As such, they are entirely compatible (as in the Susitna case) with project showing a good economic rate of return. Although only a financing characteristic brought about by the project being heavily financed by debt under inflationary conditions, this characteristic makes it possible for the project to proceed without unacceptable burden of early year costs on consumers.

(c) The Basic Financial Options

A range of financing options compatible with the conditions laid down in Senate Bill 25 have been considered as means of meeting the inflationary financing deficit. The financial parameters used in these plans are as given in subsection (a) above. The options basically consist of a range of preappropriations (in 1982) by the State of Alaska with the balance of the project financing made up by a combination of G.O. bonds and 35 year Revenue bonds, with G.O. bonds refinanced into Revenue bonds at the earliest opportunity.

(i) State of Alaska Legislative Appropriation of 100 Percent of Cost ($4.5 billion)

This conforms to the possible outcome legislated by Senate Bill 25 and represents the simplest financing option. It could take the form of the state meeting capital costs as incurred over the 15 year schedule. Alternatively, it could take the form of "preappropriation" where such a sum would be appropriated in, say 1982, as taking into account interest accumulated, would totally finance the project. For simplicity of interpretation of the options involving state appropriation all are assumed to take the "preappropriation" form.

A preappropriation of $4.5 billion in 1982 would wholly complete Susitna (on the basis of central estimates). On the basis of the present wholesale energy rate setting requirement incorporated in Senate Bill 25, the APA would, however, not be able to charge more than the actual costs incurred. Given that in this case the only costs would be the very small year-to-year operation costs, this option would involve the output from Susitna being supplied only at a fraction of the price of electricity from the best thermal option.

(ii) 50 Percent State Preappropriation ($2.5 billion) with Residual Bond Financing

The outcome for this option is summarized in terms of Figure 18.22. It is seen that it would still enable Susitna energy to be produced at a price 37 percent less than that of the best thermal option. It would also enable the project to be completed with only $1 billion (in 1982 dollars) of G.O. bonds (see below) over the period 1990-93. The Devil Canyon stage could then be completed with a further $2.4 billion (in 1982 dollars) of Revenue bonds over the period 1994 to 2002.

This level of appropriation would enable Susitna energy prices to be held virtually constant at their initial levels for nearly a decade. A temporary step-up in price to the cost of the electricity from the best thermal option would be required when Devil Canyon was completed on the basis of its 100-percent Revenue bond financing. Thereafter, however, the cost of the Susitna energy would again stabilize and give ever increasing savings compared with cost of the best thermal option.

(iii) "Minimum" State Preappropriation ($2 billion) with Residual Bond Financing

The "minimum" State appropriation is taken as the minimum amount required to meet debt service cover of 1.25 on the residual debt service cover of 1.25 on the residual debt financing by Revenue bonds and make Susitna's wholesale energy price competitive with the best thermal option in its first normal cost year (1994). This level of appropriation would require $1.8 billion (in 1982 dollars) of bond financing 1990-93 and a further $2.2 billion (in 1982 dollars) over the period 1994 to 2002 to complete Devil Canyon. (Figure 18.23).

These levels of State appropriation would all therefore eliminate Susitna's "inflationary financing deficit."

(d) Issues Arising from the Basic Financing Options

(i) Tax-Exempt Bond Financing

In the $2 billion State appropriation case interest costs on the basis of tax-exempt financing accounts for 88 percent of the unit price of Susitna output in 1994. Failure to obtain tax-exempt bond financing would increase these interest costs by approximately one third. Ensuring tax-exempt status for the Susitna bond issues is therefore of fundamental importance to the economics of the project under these options.

Difficulties could arise in obtaining tax-exempt bond financing if the financing entailed (as would probably be the case with Revenue bond financing at the precompletion stage) contracts of the take-or-pay or take-and-pay type. This is because the bulk of the

Susitna output would be taken by non-tax-exempt utilities and contracts of this type with non-tax-exempt entities would, under certain general conditions laid down in Section 103 of the IRS code, lead to the bond issues being classified as industrial development funds and foregoing their tax-exempt status. It is also questionable whether contracts with the Railbelt utilities as currently financed would constitute adequate security in the eyes of bond holders. Both these considerations indicate the need for some form of independent financing guarantee reducing dependence on the contractual relationship with the utilities.

This might take the form of the initial financing being G.O. bonds or by a State guarantee being given to the Revenue bonds. Given that either represents the same effective burden to the State, it is concluded that G.O. bonds are to be preferred on grounds of flexibility and administrative simplicity.

(ii) Refinancing Watana and the Financing of Devil Canyon

Early refinancing of any G.O. bonds used to finance Watana and the financing of Devil Canyon by Revenue bonds is taken to be an important financing objective. The main factors determining the date at which such refinancing will be possible is the magnitude of the initial state appropriation. This is dealt with in detail terms of the risk analysis in 18.5 below.

The basic conclusion from this analysis is that with a state appropriation of $2.5 billion there is a very high degree of certainty that refinancing into Revenue bonds would occur by 1994 and that the remainder of the project could be financed by Revenue bonds.

(iii) Importance of Adequate Preappropriation Funding to Subsequent Financing

The principal effect of preappropriations significantly less than $2 billion would be a possible delay in refinancing of the G.O. bonds issued to finance Watana and possibly the need for additional G.O. bond financing for Devil Canyon. This is because the impact of such lesser preappropriation would (as illustrated in Figure 18.24) give rise to inadequate earnings covered in the early years of Watana and subsequently Devil Canyon so that the raising of Revenue bonds requiring such cover had to be delayed. In addition, such inadequate funding would force the Susitna price to "track" the cost of energy from the best thermal option until adequate revenue had been obtained for such refinancing.

(iv) Impact on State Credit Rating of Susitna G.O. Bond Financing

The impact on state credit rating of G.O. bond financing of the order of $1.8 billion (in 1982 dollars) in the $2 billion state appropriation case has been assessed by the APAs financial advisors', First Boston Corporation and First Southwest Corporation. They have concurred (as fully stated in sub-section (c), (iv) of the main report) in the statement that "on the assumption that the State of Alaska's bond rating at that time is unchanged from today's level and that normality prevails in the bond market, the effect on the credit rating of the State of Alaska would not be perceptible."

(e) Conclusion

The principal conclusion of the financial evaluation is that with a state appropriation of not less than $2 billion and consent for G.O. bond financing of $1.8 billion (in 1982 dollars), Susitna would be financially viable. It would also be able to market its output at an initial price competitive with the most efficient thermal options and produce very substantial long-term savings compared with this option.

The evaluation, however, stressed the importance of establishing the project on a strong financial basis that would enable it to secure conversion of the G.O. bonds to Revenue bonds in 1994 and obtain a highly competitive rate of interest without jeopardizing the tax-exempt status of the bond issues. These objectives (together with the marketing of the Watana output in 1994 and price some 37 percent below that of the most efficient thermal option), could be secured by state appropriation of $2.5 ~~billion.~~

Methods by which the state appropriation could be recovered have not been considered since recovery is not required by existing legislation. It should be noted, however, that the cost benefit analysis shows that full recovery long-term would be possible with a better than 10-percent rate of return. Meeting the Susitna inflationary financing deficit can therefore be considered as a separate issue from subsidization of electricity prices by foregoing recovery of all or part of the State appropriation designed to meet this deficit.

18.5 - Financial Risk

The financial risks considered are those arising to the State of Alaska and to Alaskan consumers. The analysis of these risks is restricted to the period up to 2001 covering the completion of Watana and its first nine years of operation.

(a) Precompletion Risk

The major precompletion risk is the risk that the project will not be completed. The possibility of this arising owing to natural hazard is dealt with in Section _____, and on the basis of this analysis this possibility is assigned a vanishingly small probability.

The risk of non-completion owing to capital overrun is also assessed to have negligible probability. This is on the grounds that the project only involves well established technology, has been extensively assessed and surveyed, and has been assessed independently by estimators and formal probability analysis as having only a 27-to 20-percent probability of any real capital overrun.

(b) Post-Completion Risks

(i) The Generation of Post-Completion Risks

A probabilistic financial model was developed taking into account the probability distributions of the major engineering and financial variables on which the financial outcome for Susitna depends. This model was then used to consider in detail critical specific risks and the aggregative risk posed by the project.

(ii) Specific Risks

- Specific Risk I; Risk of G.O. Bond Financing Overrun (Figure 18.25)

Extensive analysis was undertaken to assess the probability that the G.O. bond financing requirements would overrun the forecast values as a result of capital costs, inflation, interest rates, etc., being less favorable than forecast. In the $2.5 billion state appropriation case it was found that the probability of the G.O. bond requirement exceeding the forecast of $1 billion (in 1982 dollars) by more than 50 percent was only 0.15. This implies that there is less than one in six chance of the G.O. bond overrun exceeding $1.5 billion. The probability of its exceeding $2 billion was only 0.03. There is also a significant probability that the bond financing requirements will be less than forecast.

- Specific Risk II; Delayed Conversion of G.O. Bonds

Minimization of the magnitude and duration of G.O. bond requirements is taken as an important financial objective. Evaluation of this specific risk in the $2.5 billion appropriation case indicates that:

. The probability of any delay compared with the forecast date of 1994 for the conversion from G.O. bonds to Revenue bonds is 0.05.

. The latest date at which complete conversion to Revenue bonds occurs in any outcome is 1996 (3 years after completion of Watana).

- Specific Risk III: Early Year Non-viability (Figure 18.26)

The measure of financial non-viability in the early years is taken as the ratio of Watana's unit cost to the costs of the best thermal option in Watana's third year (1996). (For comparability debt service excess cover was excluded). This analysis indicates that there is only a 0.15 chance of the Susitna costs exceeding their forecast value (30 percent of the best thermal) by more than 15 percentage points.

(iii) The Aggregate Risk

While specific risks of the type considered above are of importance basic concern must center on the aggregate risk. In long-term economics this is measured by the risk attaching to the rate of return. For the purpose of the financial risk, however, it is taken as represented by accumulative net operating earnings at the end of the first nine years of operation of Watana. Since this statistic is net of interest and debt repayment, if effectively subsumes all the risks involved in capital expenditure, inflation, interest rates, revenue, etc., deviating from their forecast values. This statistic was also adjusted to allow the pricing up of Watana energy to the cost of the best thermal option so that statistic affects the "upside" risk as well as the "downside."

On this basis the statistic (see Figure 18.27) was found to have only a 0.15 chance of being below forecast level of $1.35 billion (in 1982 dollars) by more than $.35 billion. There is also a 0.34 probability of the statistic exceeding $1.5 billion and thus creating greater savings for the Alaskan consumer.

(c) Conclusions

The analysis shows the exposure of the project either to critical specific risks or to aggregative risk is relatively limited. The qualification attaching to this analysis is that the estimates and probabilities used are free from any systematic biases. The structure of the plan of the overall plan of study for Susitna and analysis of its alternatives has however been specifically designed to take every reasonable precaution against this possibility by seeking extensive independent verification of the key variables by Batelle and Ebasco operating wholly as independent consultants.

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TABLE 18.1: REAL (INFLATION-ADJUSTED) ANNUAL GROWTH IN OIL PRICES

	Growth Rates (Percent)		
	1982-2000	2000-2040	Probability
Low Case	0	0	0.3
Medium (most likely case)	2.0	1.0	0.5
High Case	4.0	2.0	0.2

Base Period
(January 1982)

Price of No. 2 Fuel Oil - $6.50/MMBtu.

TABLE 18.2: DOMESTIC MARKET PRICES AND EXPORT OPPORTUNITY VALUES OF NATURAL GAS

	Domestic Market Price[1]			Export Opportunity Value		
	Low	Medium	High	Low	Medium	High
Probability of Occurrence	N.A.	N.A.	N.A.	27%	46%	27%
Base Period Value		$3.00/MMBtu			$4.65/MMBtu[2]	
Real Escalation CIF Price, Japan[3]						
1982 - 2000		N.A.		0%	2%	4%
2000 - 2040				0%	1%	2%
Real Escalation Alaska Price[4]						
1982 - 2000	0%	2.5%	5.0%	0%	2.7%	5.2%
2000 - 2040	0%	2.0%	2.0%	0%	1.2%	2.2%

1 OGP analysis used domestic market prices with zero escalation beyond 2010. (Source: Battelle)

2 Based on CIF prce in Japan ($6.75) less estimated cost of liquefaction and shipping ($2.10). (Source: 13, 14, 15).

3 Source: (9), (16).

4 Alaska opportunity value escalates more rapidly than CIF prices as liquefaction and shipping costs are estimated to remain constant in real terms.

TABLE 18.3: SUMMARY OF COAL OPPORTUNITY VALUES

	Base Period (Jan. 1982) Value ($/MMBtu)	Annual Real Grwoth Rate 1980 - 2000 (%)	2000 - 2040 (%)	Probability of Occurrence
Base Case				
Battelle Base Period CIF Price				
Medium Scenario				
- CIF Japan	1.95	2.0	1.0	49
- FOB Beluga	1.43	2.6	1.2	49
- Nenana	1.75	2.3	1.1	49
Low Scenario				
- CIF Japan	1.95	0	0	24
- FOB Beluga	1.43	0	0	24
- Nenana	1.75	0.1	0.1	24
High Scenario				
- CIF Japan	1.95	4.0	2.0	27
- FOB Beluga	1.43	5.0	2.2	27
- Nenana	1.75	4.5	1.9	27
Sensitivity Case				
Updated Base Period CIF Price[1]				
Medium Scenario				
- CIF Japan	2.66	2.0	1.0	49
- FOB Beluga	2.08	2.5	1.2	49
- FOB Nenana	1.74	2.7	1.2	49
Low Scenario				
- CIF Japan	2.66	0	0	24
- FOB Beluga	2.08	0	0	24
- FOB Nenana	1.74	-0.2	-0.1	24
High Scenario				
- CIF Japan	2.66	4.0	2.0	27
- FOB Beluga	2.08	4.8	2.2	27
- FOB Nenana	1.74	5.3	2.3	27

1 Assuming a 10 percent discount for Alaskan coal due to quality differentials, and export potential for Heal coal.

TABLE 18.4: SUMMARY OF FUEL PRICES USED IN THE OGP PROBABILITY TREE ANALYSIS

	Fuel Price Scenario		
	Low	Medium	High
Probability of occurrence	25%	50%	25%
Base period January 1982 prices			
(1982$/MMBtu)			
Fuel Oil	6.50	6.50	6.50
Natural Gas	3.00	3.00	3.00
Coal			
- Beluga	1.43	1.43	1.43
- Nenana	1.75	1.75	1.75
Real escalation rates per year (percent)[1]			
Fuel Oil			
- 1982 - 2000	0	2.0	4.0
- 2000 - 2040	0	2.0	2.0
Natural Gas			
- 1982 - 2000	0	2.5	5.0
- 2000 - 2040	0	2.0	2.0
Beluga Coal			
- 1982 - 2000	0	2.6	5.0
- 2000 - 2040	0	1.2	2.2
Nenana Coal			
- 1982 - 2000	0.1	2.3	4.5
- 2000 - 2040	0.1	1.1	1.9

1 Beyond 2010, the OGP analysis has used zero real escalation in all cases.

TABLE 18.5: ECONOMIC ANALYSIS
SUSITNA PROJECT - BASE PLAN

			1982 Present Worth of System Costs $ x 10^6			
Plan	ID	Components	1993-2010	2010	Estimated 2011-2051	1993-2051
Non Susitna	A	600 MW Coal-Beluga	3,213	491	5,025	8,238
		200 MW Coal-Nenana				
		630 MW GT				
Susitna	C	680 MW Watana	3,119	385	3,943	7,062
		600 MW Devil Canyon				
		180 MW GT				
Net Economic Benefit of Susitna Plan						1,716

TABLE 18.6: SUMMARY OF LOAD FORECASTS USED FOR SENSITIVITY ANALYSIS

	Medium		Low		High	
	MW	GWh	MW	GWh	MW	GWh
1990	892	4,456	802	3,999	1,098	5,703
2000	1,084	5,469	921	4,641	1,439	7,457
2010	1,537	7,791	1,245	6,303	2,165	11,435

TABLE 18.7: LOAD FORECAST SENSITIVITY ANALYSIS

			1982 Present Worth of System Costs ($ x 10^6)				
Plan	ID	Components	1993-2010	2010	Estimated 2011-2051	1993-2051	Net Economic Benefit
Non-Susitna	K_1	400 MW Coal-Beluga	2,640	404	4,238	6,878	-
Low Forecast		200 MW Coal-Nenana 560 MW GT					
Susitna	K_2	680 MW Watana (1995)	2,882	360	3,768	6,650	228
Low Forecast		600 MW Devil Canyon (2004)					
Non-Susitna	J_1	800 MW Coal-Beluga	4,176	700	6,683	10,8591[1]	-
High Forecast		200 MW Coal-Nenana 700 MW GT 430 MW Pre-1993					
Susitna	J_2	680 MW Watana (1993)	3,867	564	5,380	9,2471[1]	1,612
High Forecast		600 MW Devil Canyon (1997) 350 MW GT 430 MW Pre-1993					

[1] From 1993 to 2040

TABLE 18.8: DISCOUNT RATE SENSITIVITY ANALYSIS

Plan	ID	Real Discount Rate (Percent)	1982 Present Worth of System Costs ($ x 10^6) 1993-2010	2010	Estimated 2011-2051	1993-2051	Net Economic Benefit
Non-Susitna	Q_1	2	3,701	465	7,766	11,167	-
Susitna	Q_2	2	3,156	323	5,394	8,550	2,617
Non-Susitna	A	3	3,213	491	5,025	8,328	-
Susitna	C	3	3,119	385	3,943	7,062	1,176
Non-Susitna	S_1	4	2,791	517	3,444	6,235	-
Susitna	S_2	4	3,080	457	3,046	6,126	109
Non-Susitna	P_1	5	2,468	550	2,478	4,946	-
Susitna	P_2	5	3,032	539	2,426	5,459	(513)

TABLE 18.9: CAPITAL COST SENSITIVITY ANALYSIS

Plan	ID	1982 Present Worth of System Costs $ x 10^6: 1993-2010	2010	Estimated 2011-2051	1993-2051	Net Economic Benefit
Non-Susitna Capital Costs Up 20 Percent						
Non-Susitna	G	3,460	528	5,398	8,858	-
Susitna	C[1]	3,119	385	3,943	7,062	1,976
Non-Susitna Capital Costs Down 10 Percent						
Non-Susitna	G	3,084	472	4,831	7,915	-
Susitna	C[1]	3,119	385	3,943	7,062	853
Susitna Capital Costs Less Contingency						
Non-Susitna	A	3,213	491	5,025	8,238	-
Susitna	X_2	2,710	336	3,441	6,151	2,087
Susitna Capital Costs Plus Doubled Contingency						
Non-Susitna	A	3,213	491	5,025	8,238	-
Susitna	Y_2	3,529	434	4,445	7,974	264

[1] An adjustment calculation was made regarding the ± capital costs of the 3GT units added in 2007-2010 since the difference was less than $\$10 \times 10^6$. Beyond 2010, this effect was not included.

TABLE 18.10: SENSITIVITY ANALYSIS - UPDATED BASE PLAN (JANUARY 1982) COAL PRICES

	Base Period Beluga Coal Price (1982 $/MMBtu)	PW in 1982 ($ x 10^6)		
		Costs of Non-Susitna Plan	Costs of Susitna Plan	Net Economic Benefits
Base Case	1.43	8,238	7,062	1,176
Sensitivity (Updated) Case	2.08	--	--	--

TABLE 18.12: SENSITIVITY ANALYSIS - REAL COST ESCALATION

Plan	ID	1993-2010	2010	Estimated 2011-2051	1993-2051	Net Benefit
		1982 Present Worth of System Costs ($ X 10^6)				
Zero-Escalation in Capital and O&M Costs						
. Non-Susitna	O_1	2,838	422	4,319	7,157	-
. Susitna	O_2	2,525	299	3,060	5,585	1,572
Double Escalation Capital and O&M Costs						
. Non-Susitna	P_1	3,650	602	6,161	9,811	-
. Susitna	P_2	3,881	503	5,148	9,029	782
Zero-Escalation in Fuel Prices						
. Non-Susitna	V_1	2,233	335	3,427	5,660	-
. Susitna	V_2	3,002	365	3,736	6,738	(1,078)
High Escalation in Fuel Prices						
. Non-Susitna	W_1	4,063	643	6,574	10,367	-
. Susitna	W_2	3,267	403	4,121	7,388	2,979

TABLE 18.12(a): SENSITIVITY ANALYSIS - NON-SUSITNA PLAN WITH CHACKACHAMNA

Plan	ID	Components	1993 2010	2010	Estimated 2011-2051	1993-2051	Net Benefit
			1982 Present Worth of System Costs ($ X 10^6)				
. Non-Susitna with Chackachamna	B	330 MW Chackachamna 400 MW Coal-Beluga 200 MW Coal-Nenana 440 MW GT	2,038	475	4,861	7,899	-
. Susitna	C	680 MW Watana 600 MW Devil Canyon 180 MW GT	3,119	385	3,943	7,062	837

TABLE 18.13: SUMMARY OF SENSITIVITY ANALYSIS INDEXES OF NET ECONOMIC BENEFITS BASE CASE ($1,176 MILLION) = 100

Fuel Escalation	
- High	253[1]
- Low	-92[2]
Discount Rates	
- High-High (5%)	-44
- High (4%)	9
- Low (2%)	223
Susitna Capital Cost	
- High	23
- Low	178
Load Forecast	
- High	137
- Low	19
Non-Susitna (Thermal) Capital Costs	
- High	168
- Low	73
Capital and O&M Cost Escalation	
- High	67
- Low	134
Chackachamna (included in Non-Susitna Plan)	71
Updated Base Coal Price	

[1] High fuel escalation case provides net benefits equal to 253 percent of the base value, 2.53 x 1,176, or 2,975.

[2] Low fuel escalation case provides minus 92 percent of the base case net benefits, -.92 x 1,176, or -1,082.

UTILITY	Generating Capacity 1981 MW at 0°F Rating	Predominant Type of Generation	Tax Status Re: IRS Section 103	Purchases Wholesale Electrical Energy	Provides Wholesale Supply	Utility Annual Energy Demand 1980 GWh
IN ANCHORAGE-COOK INLET AREA						
Anchorage Municipal Light and Power	221.6	SCCT	Exempt	*	–	585.8
Chugach Electric Association	395.1	SCCT	Non-Exempt	*	*	941.3
Matanuska Electric Association	0.9	Diesel	Non-Exempt	*	–	268.0
Homer Electric Association	2.6	Diesel	Non-Exempt	*	–	284.8
Seward Electric System	5.5	Diesel	Non-Exempt	*	–	26.4
Alaska Power Administration	30.0	Hydro	Non-Exempt	–	*	–
National Defense	58.8	ST	Non-Exempt	–	–	–
Industrial – Kenai	25.0	SCCT	Non-Exempt	–	–	–
IN FAIRBANKS – TANANA VALLEY						
Fairbanks Municipal Utility System[1]	68.5	ST/Diesel	Exempt	–	–	116.7
Golden Valley Electric Association[1]	221.6	SCCT/Diesel	Non-Exempt	–	–	316.7
University of Alaska	18.6	ST	Non-Exempt	–	–	–
National Defense[1]	46.5	ST	Non-Exempt	–	–	–
IN GLENALLEN/VALDEZ AREA						
Copper Valley Electric Association	19.6	SCCT	Non-Exempt	–	–	37.4
TOTAL	1114.3					2577.1

[1]Pooling Arrangements in Force

TABLE 18.14 – RAILBELT UTILITIES PROVIDING MARKET POTENTIAL

ACRES

PLANT LIST

PLANT No.	NAME OF PLANT	UTILITY	TYPE OF OWNERSHIP
2	Anchorage No. 1	Anchorage Municipal Light and Power	Municipal
3	Anchorage	Anchorage Municipal Light and Power	Municipal
6	Eklutna	Alaska Power Administration	Federal
7	Chena	Fairbanks Municipal Utilities System	Municipal
10	Knik Arm	Chugach Electric Association, Inc.	Cooperative
22	Elmendorf-West	United States Air Force	Federal
2[illegible]	Fairbanks	Golden Valley Electric Association, Inc.	Cooperative
32	Cooper Lake	Chugach Electric Association, Inc.	Cooperative
34	Elmendorf-East	United States Air Force	Federal
35	Ft. Richardson	United States Army	Federal
36	Ft. Wainright	United States Air Force	Federal
37	Eielson	United States Air Force	Federal
38	Ft. Greeley	United States Army	Federal
47	Bernice Lake	Chugach Electric Association, Inc.	Cooperative
55	International Station	Chugach Electric Association, Inc.	Cooperative
58	Healy	Golden Valley Electric Association, Inc.	Cooperative
59	Beluga	Chugach Electric Association, Inc.	Cooperative
75	Clear AFB	United States Air Force	Federal
80	Collier-Kenai	Collier-Kenai	Municipal
81	Eyak	Cordova Public Utilities	Municipal
82	North Pole	Golden Valley Electric Association, Inc.	Cooperative
83	Valdez	Golden Valley Electric Association, Inc.	Cooperative
84	Glennallen	Golden Valley Electric Association, Inc.	Cooperative

TABLE 18.15 – LIST OF GENERATING PLANT SUPPLING RAILBELT REGION

ACRES

TABLE 18.16: ESTIMATED FINANCIAL PARAMETERS

	Watana	Devil Canyon	Total
Project Completion - Year	1993	2002	
Energy Level - 1993			3 387 GW·h
- 2002			5 721 "
- 2010			6 616 "
Costs in January 1982 Dollars			
Capital Costs	$ 3.647 billion	$1.470 billion	$ 5.117 billion
Operating Costs - per annum	$10.0 million	$5.42 million	$15.42 million
Provision for Capital			
Renewals - per annum (0.3 percent of Capital Costs)	$10.94 million	$4.41 million	$15.35 million

Operating Working Capital
- 15 percent of Operating Costs
- 10 percent of Revenue

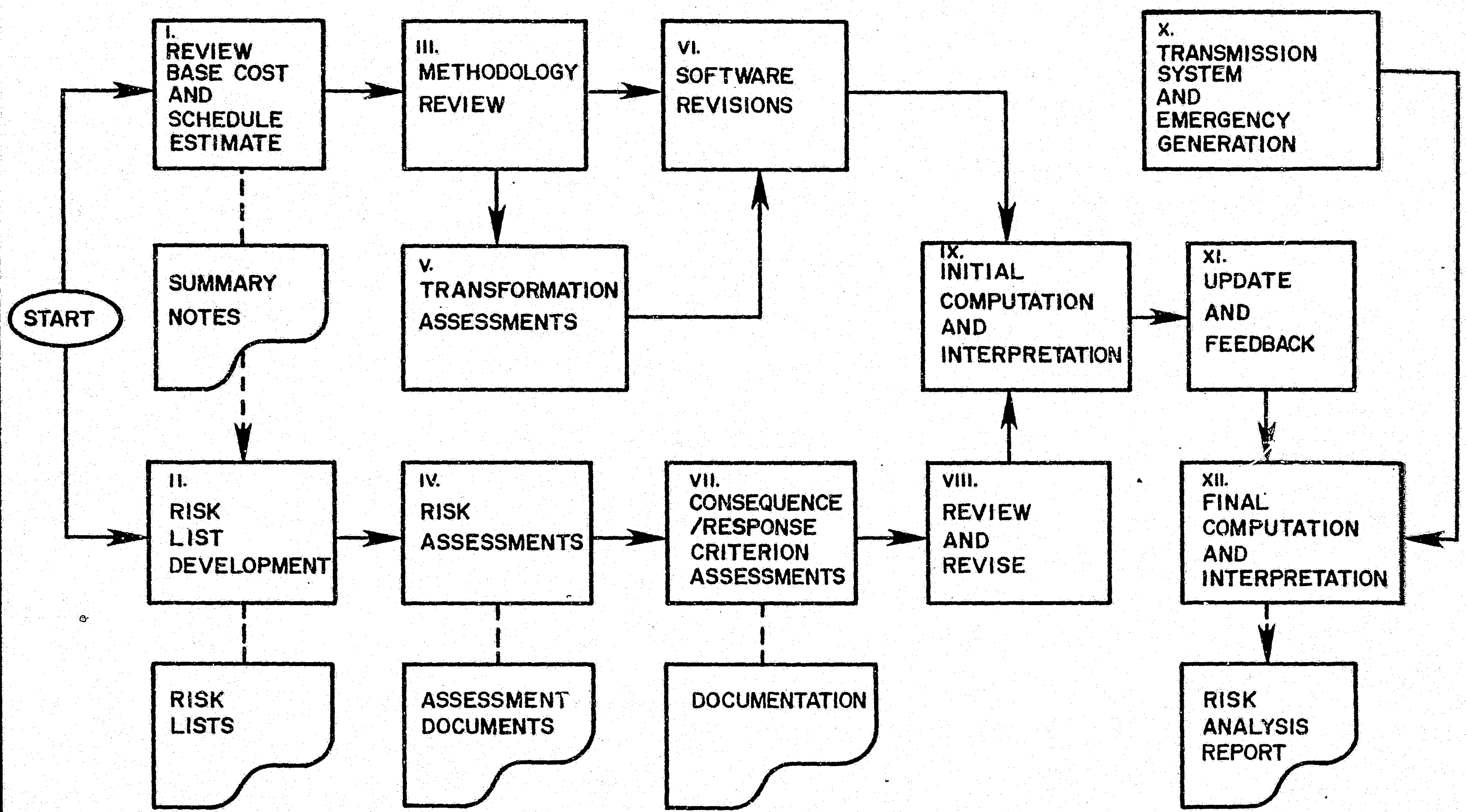
Reserved Contingency Fund
- 100 percent of Operating Costs
- 100 percent of Provision for Capital Renewals

Interest Rate - 10 percent per annum
Debt Repayment Period - 35 years

Inflation Rate - 7 percent per annum

Real Increase in Operating Costs
- 1982 to 1987 - 1.7 percent per annum
- 1988 on - 2.0 percent per annum

Real Increase in Capital Costs
- 1982 to 1985 - 1.1 percent per annum
- 1986 to 1992 - 1.0 percent per annum
- 1993 on - 2.0 percent per annum



RISK ANALYSIS STUDY METHODOLOGY

ACRES

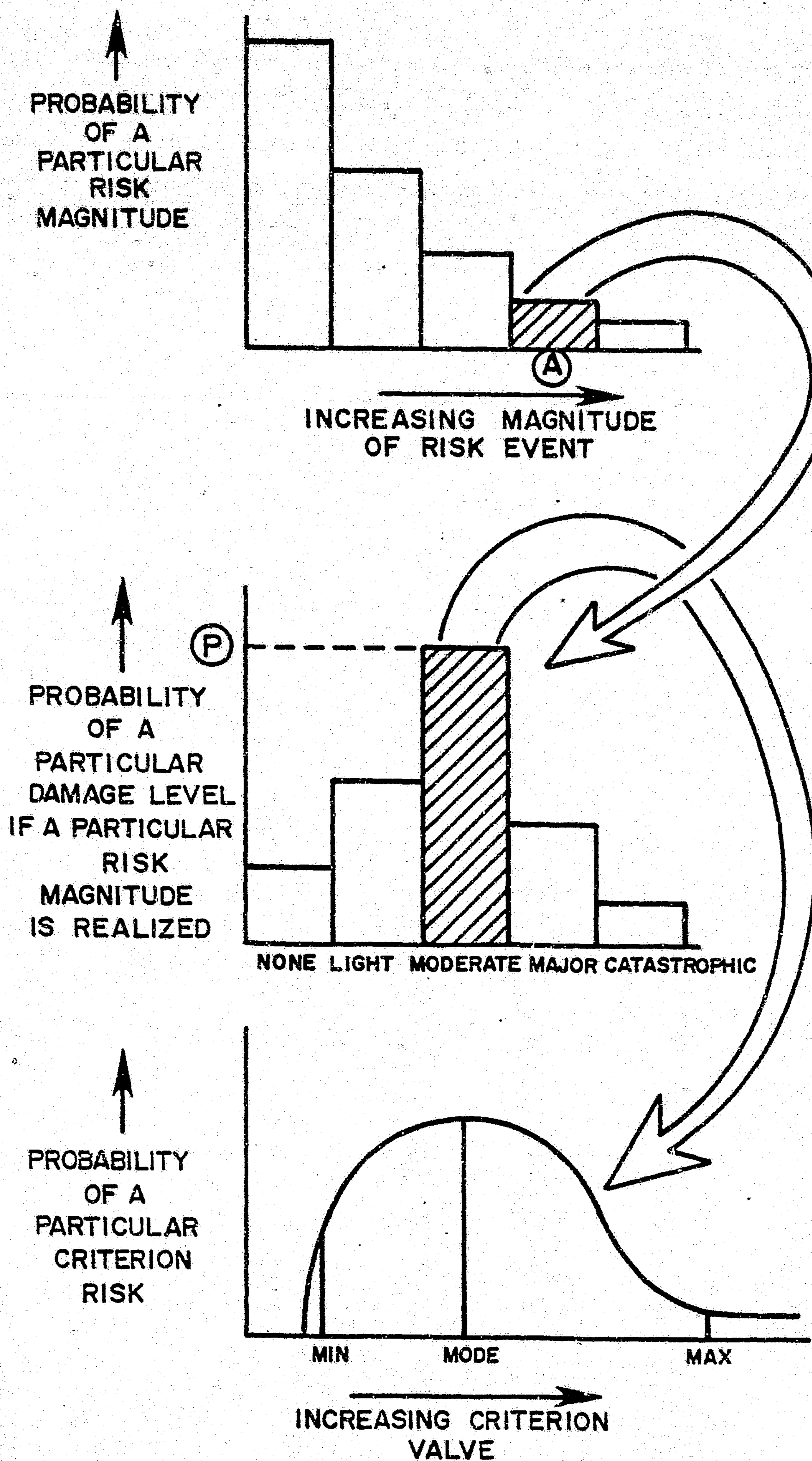
FIGURE 18.1

QUESTION:	STUDY ELEMENT:
WHAT MAJOR CONSTRUCTION PROJECTS ARE INVOLVED?	CONFIGURATIONS
WHAT KIND OF WORK IS GOING ON FOR A GIVEN CONFIGURATION?	CONFIGURATION STATES
WHAT ARE THE POSSIBLE INITIATING MECHANISMS WHICH COULD INFLUENCE ESTIMATED COSTS OR COMPLETION TIMES?	RISKS
WHAT MAJOR PORTIONS OF ANY GIVEN CONFIGURATION ARE SUBJECT TO RISK REALIZATION?	ACTIVITIES
IF A PARTICULAR RISK MAGNITUDE IS REALIZED, WHAT POSSIBLE CONSEQUENCES CAN OCCUR?	DAMAGE SCENARIOS
HOW CAN THESE CONSEQUENCES BE MEASURED?	CRITERIA
WHAT IMPORTANT ASSUMPTIONS AND LIMITATIONS MUST BE ESTABLISHED TO PERMIT A REASONABLE ANALYSIS AND TO DRAW IMPORTANT CONCLUSIONS?	BOUNDARY CONDITIONS

ELEMENTS OF THE RISK ANALYSIS

FIGURE 18.2

ACRES



(1) A SERIES OF DISCRETE RISK PROBABILITY LEVELS EXISTS FOR EACH RISK-ACTIVITY COMBINATION. THE ANNUAL PROBABILITY OF EACH IS DETERMINED.

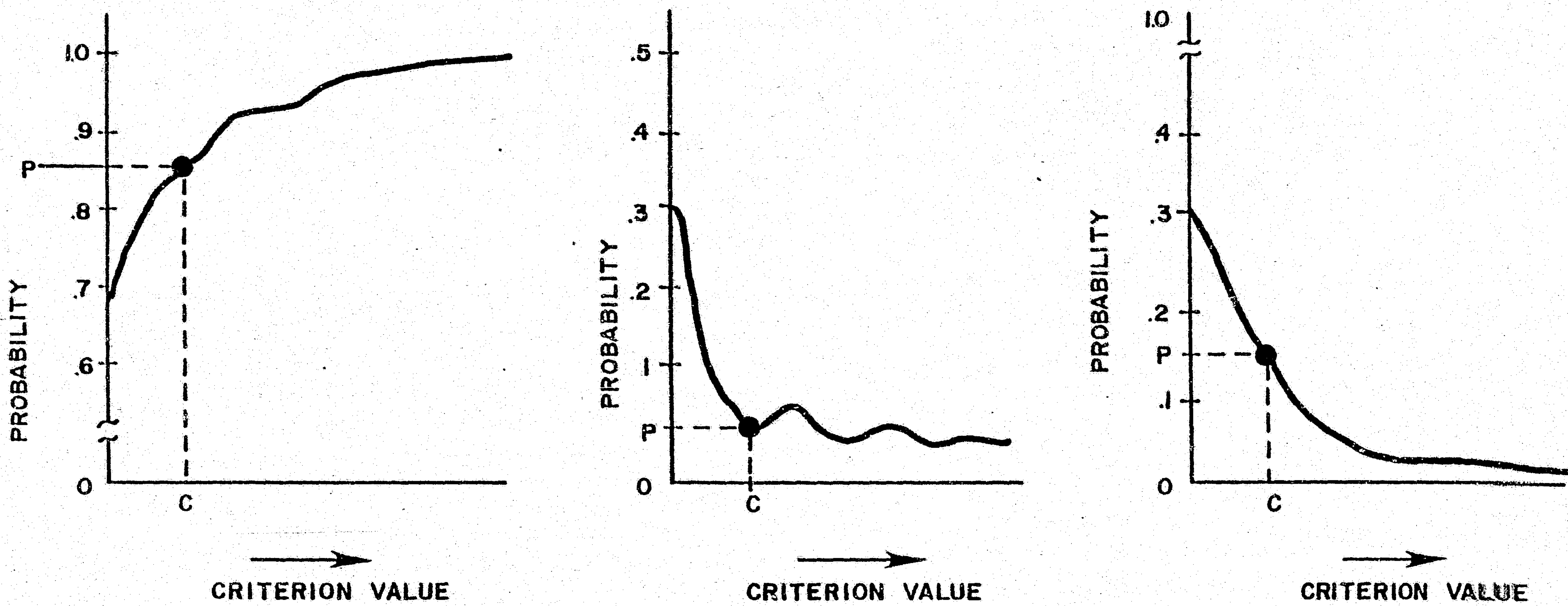
(2) IF A RISK EVENT OCCURS, IT CAN CAUSE A NUMBER OF POSSIBLE DAMAGE LEVELS, EACH WITH A PARTICULAR PROBABILITY OF OCCURENCE. IF RISK MAGNITUDE (A) OCCURS, THE PROBABILITY IT WILL CAUSE MODERATE DAMAGE IS THE VALUE OF (P) ON THE DIAGRAM.

(3) FOR ANY GIVEN DAMAGE LEVEL, THREE CRITERION VALVES ARE ESTIMATED AND FIT TO A MODIFIED BETA DISTRIBUTION.

STRUCTURAL RELATIONSHIP FOR HANDLING RISK ACTIVITY COMBINATIONS, DAMAGE SCENARIOS AND CRITERION VALUES

ACRES

FIGURE 18.3



(1) CUMULATIVE DISTRIBUTION

ANY POINT ON THE CURVE INDICATES THE PROBABILITY (P) THAT THE CRITERION VALUE (C) WILL NOT BE EXCEEDED.

(2) DENSITY FORM

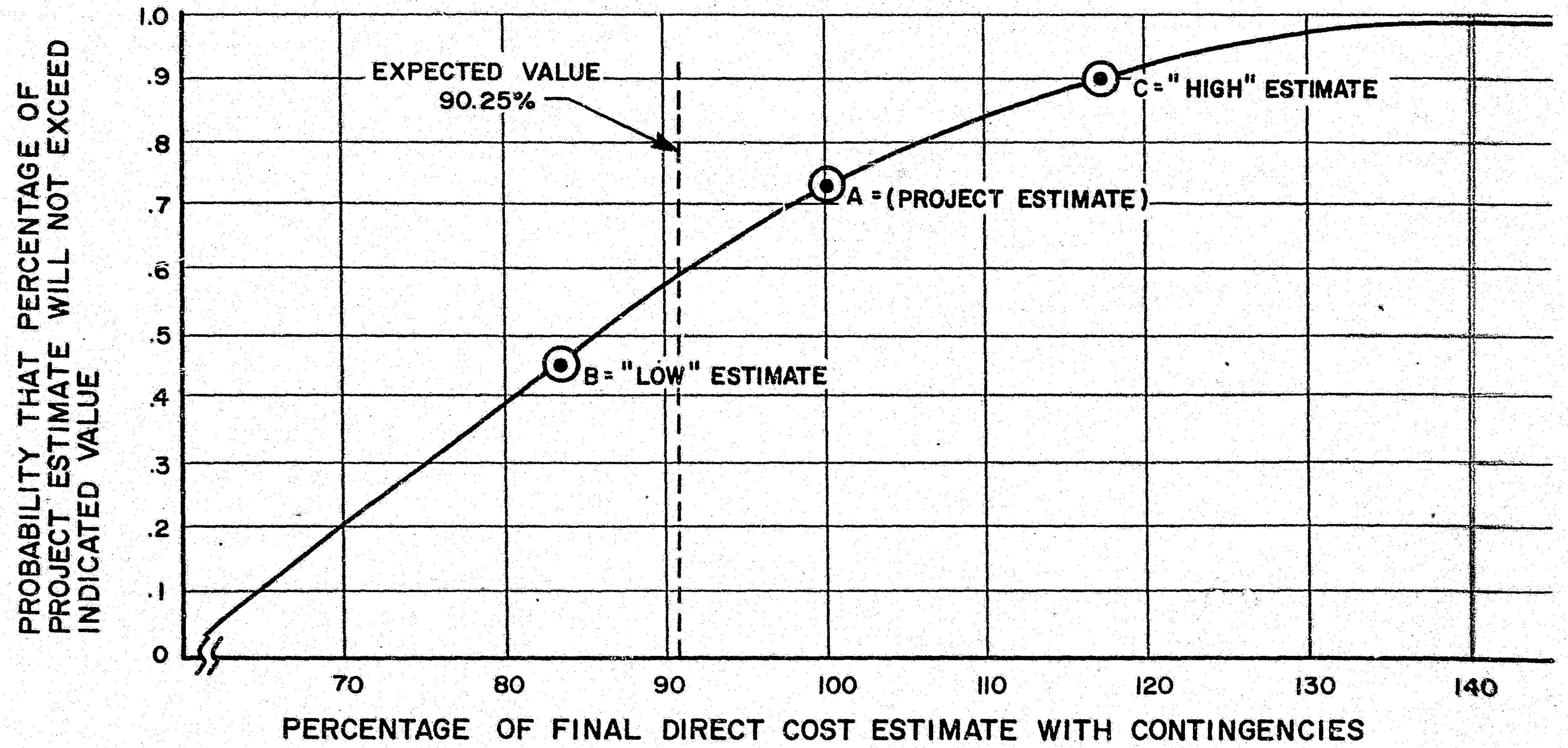
ANY POINT ON THE CURVE INDICATES THE PROBABILITY (P) THAT A PARTICULAR CRITERION VALUE (C) WILL BE INCURRED.

(3) REVERSE CUMULATIVE

ANY POINT ON THE CURVE INDICATES THE PROBABILITY (P) THAT THE CRITERION VALUE (C) WILL BE EXCEEDED.

ALTERNATIVE FORMATS FOR PRESENTING THE ANALYTICAL RESULTS

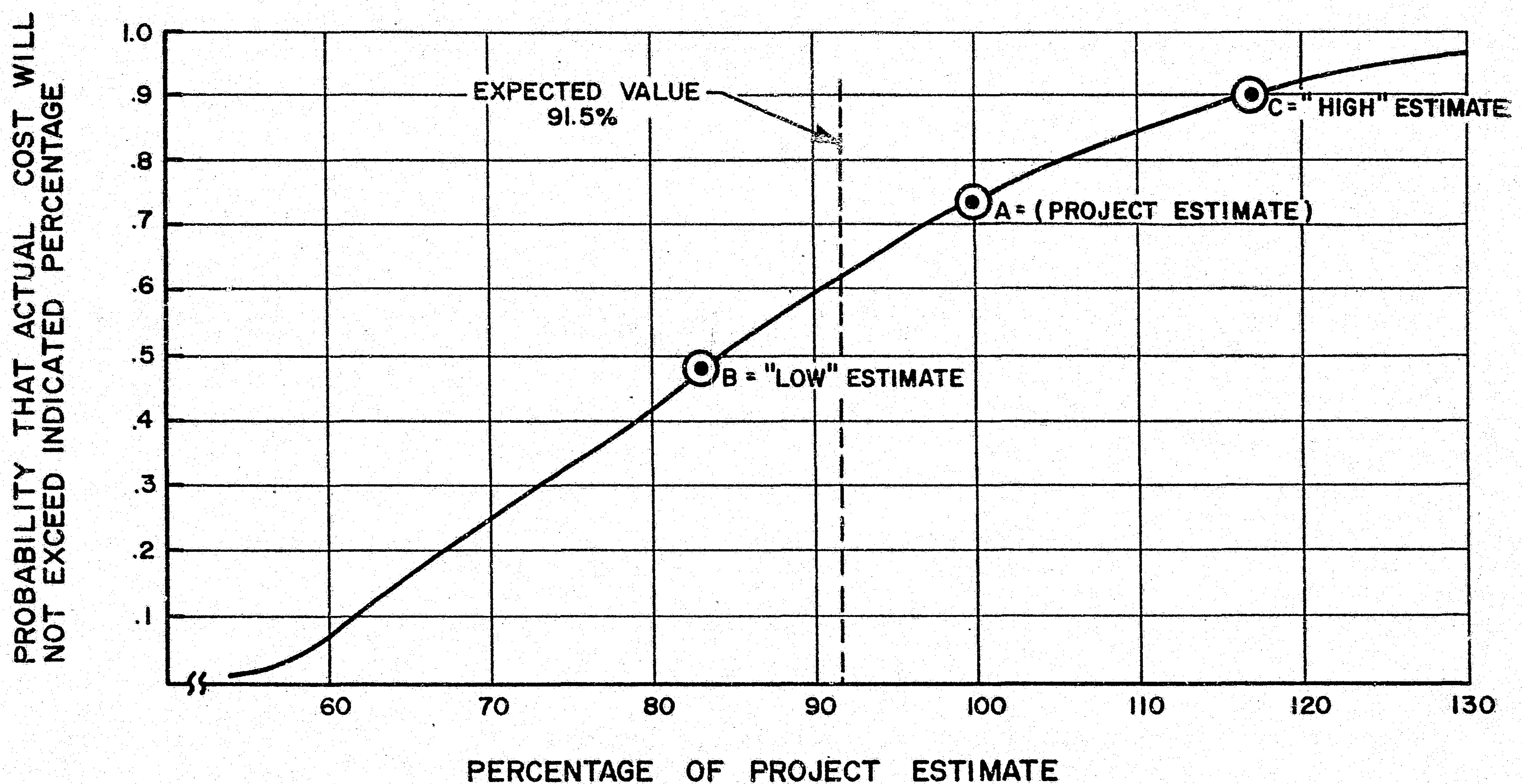
FIGURE 18.4



CUMULATIVE PROBABILITY DISTRIBUTION
FOR WATANA PROJECT COST

FIGURE 18.5

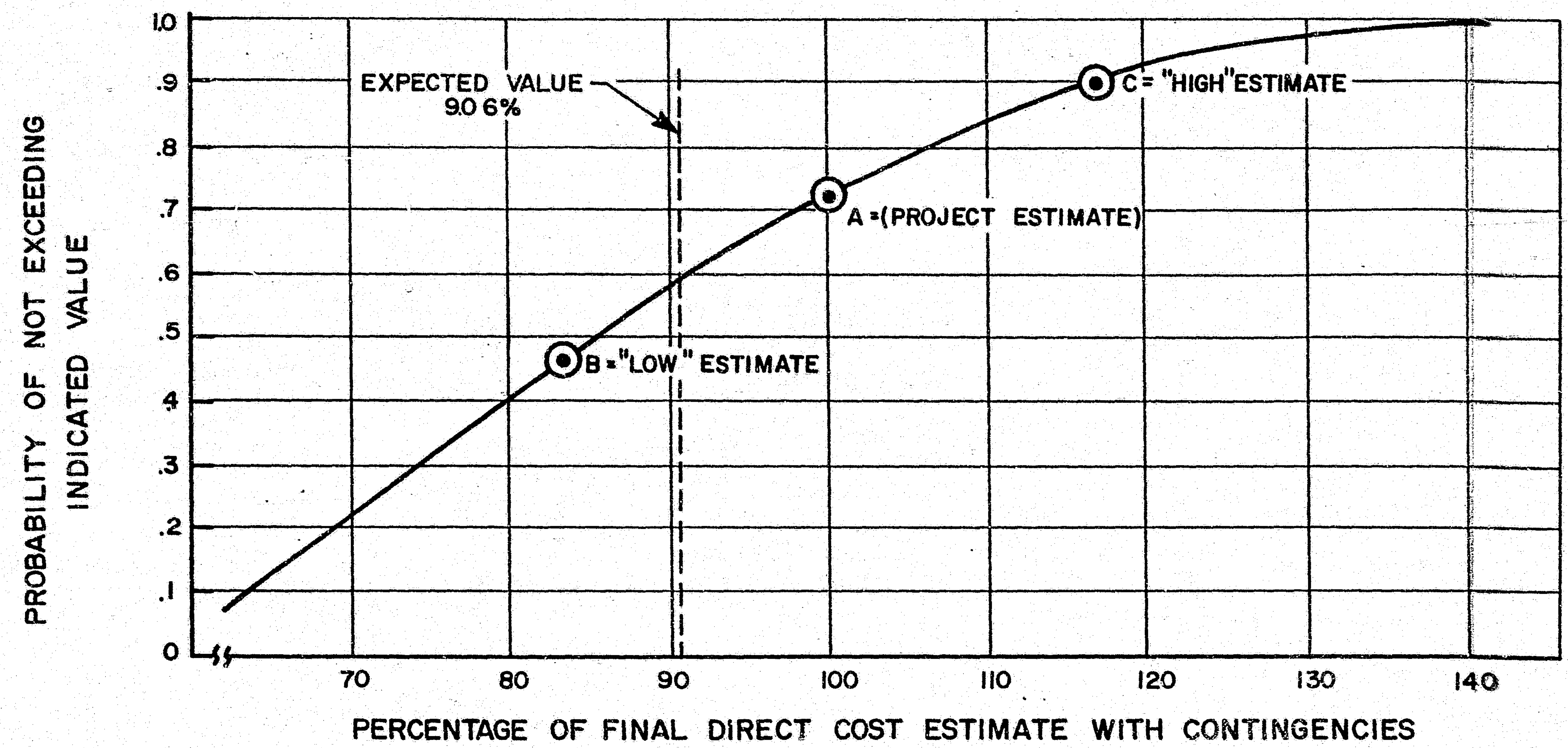
ACRES



CUMULATIVE DISTRIBUTION
OF DEVIL CANYON COSTS

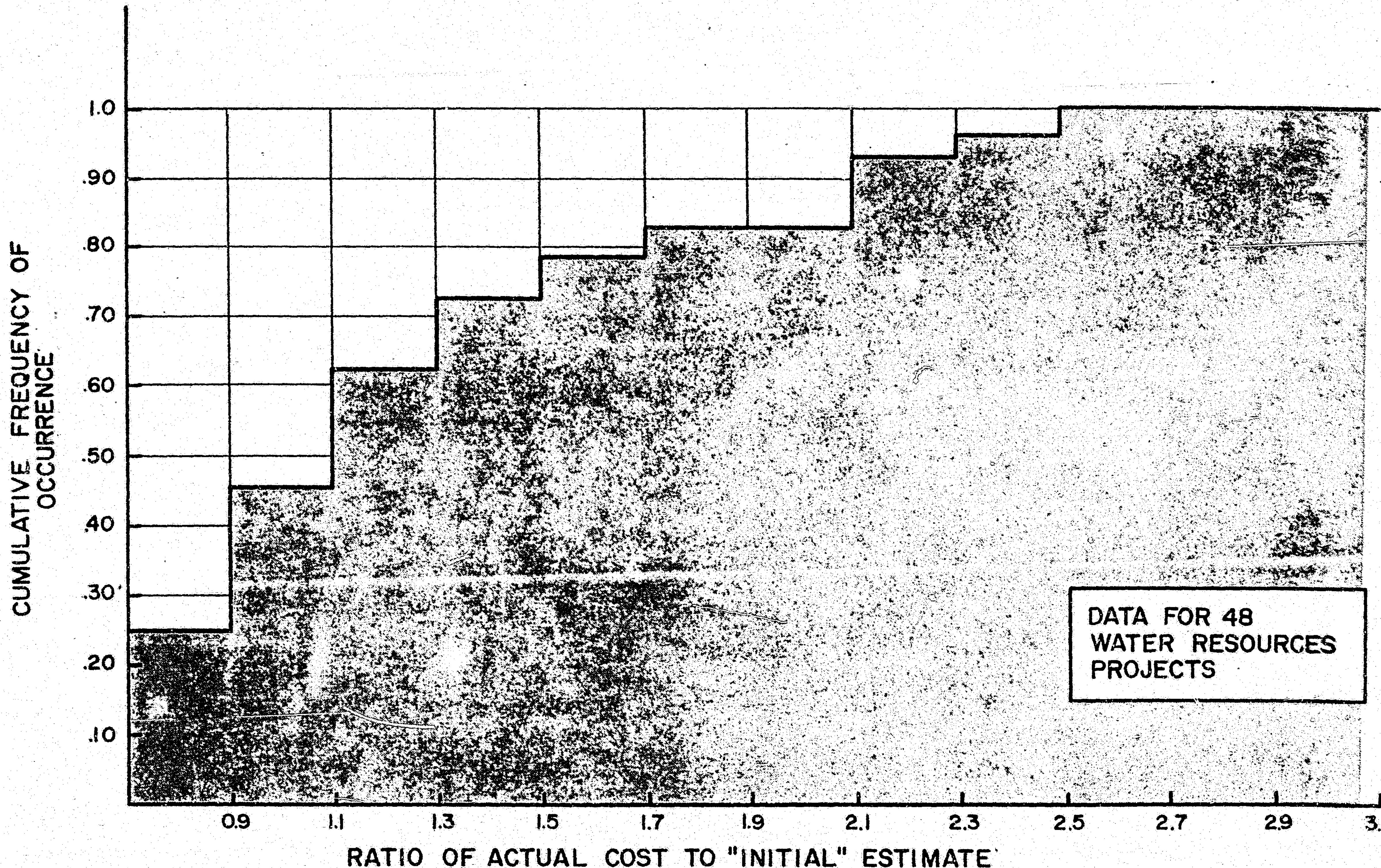
ACRES

FIGURE 18.6



CUMULATIVE PROBABILITY DISTRIBUTION
FOR SUSITNA HYDROELECTRIC PROJECT

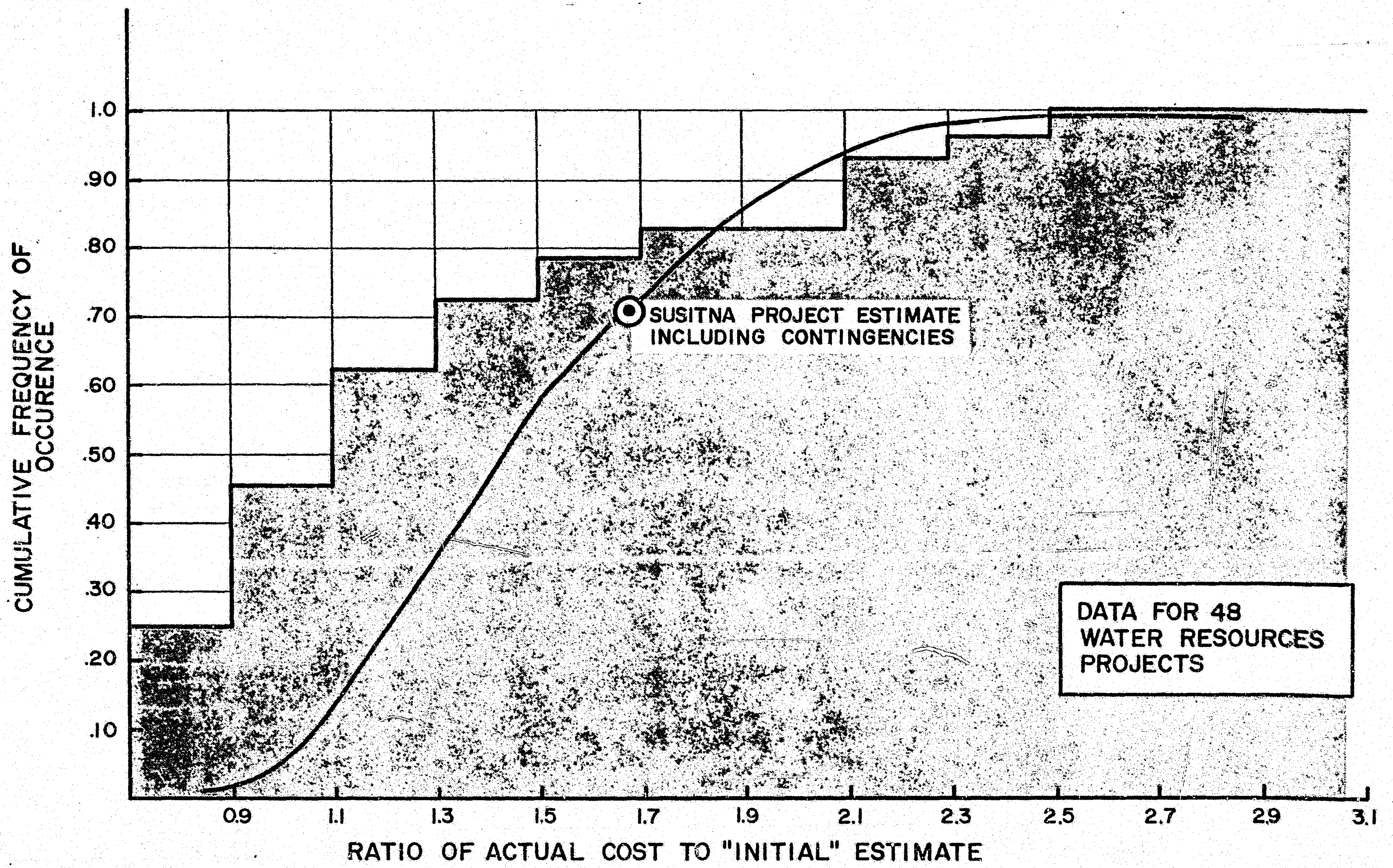
FIGURE 18.7



HISTORICAL WATER RESOURCES
PROJECT COST PERFORMANCE (48 PROJECTS)

ACRES

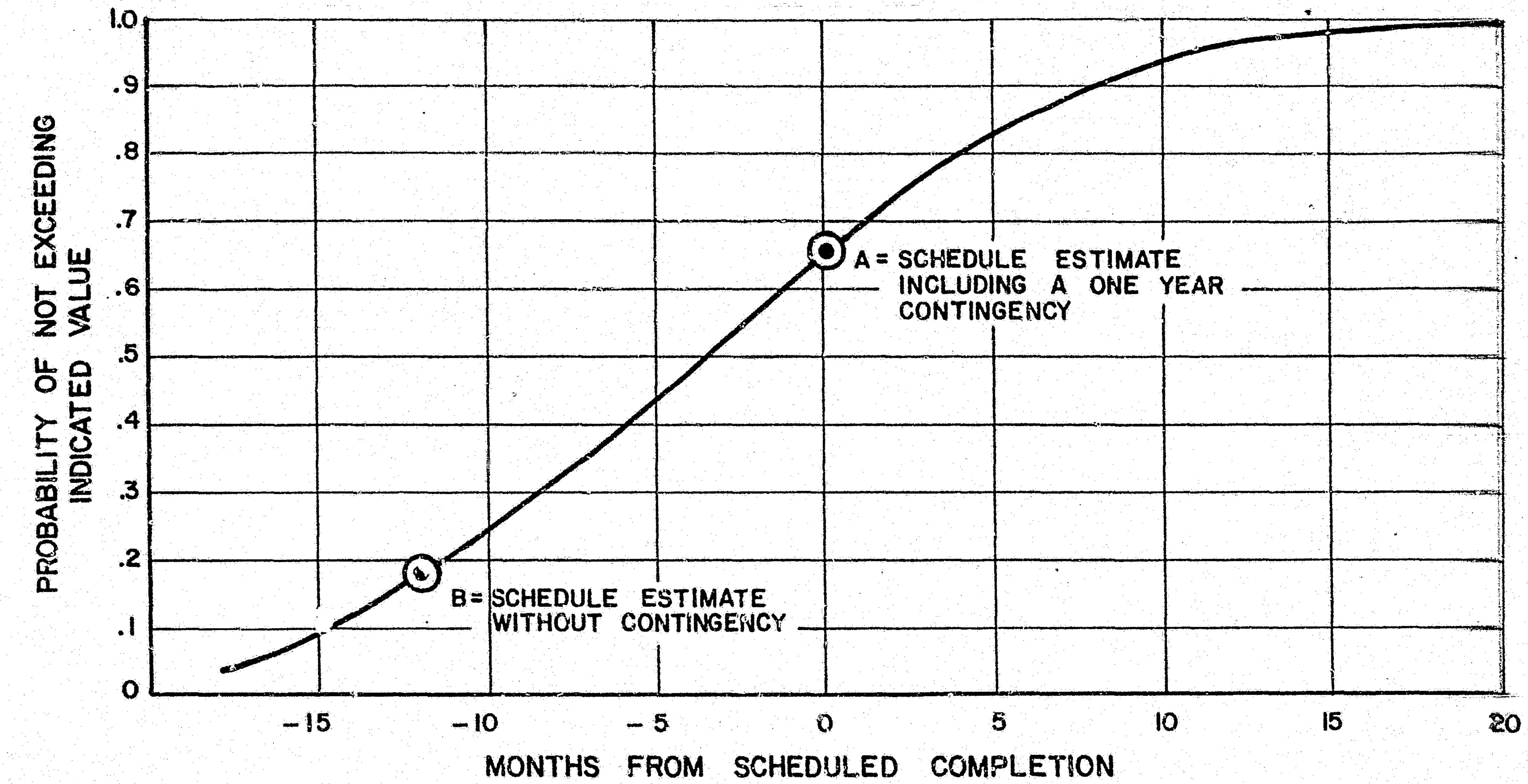
FIGURE 18.8



COMPARISON OF SUSITNA RISK RESULTS WITH HISTORICAL WATER RESOURCES PROJECT COST PERFORMANCE (48 PROJECT)

ACRES

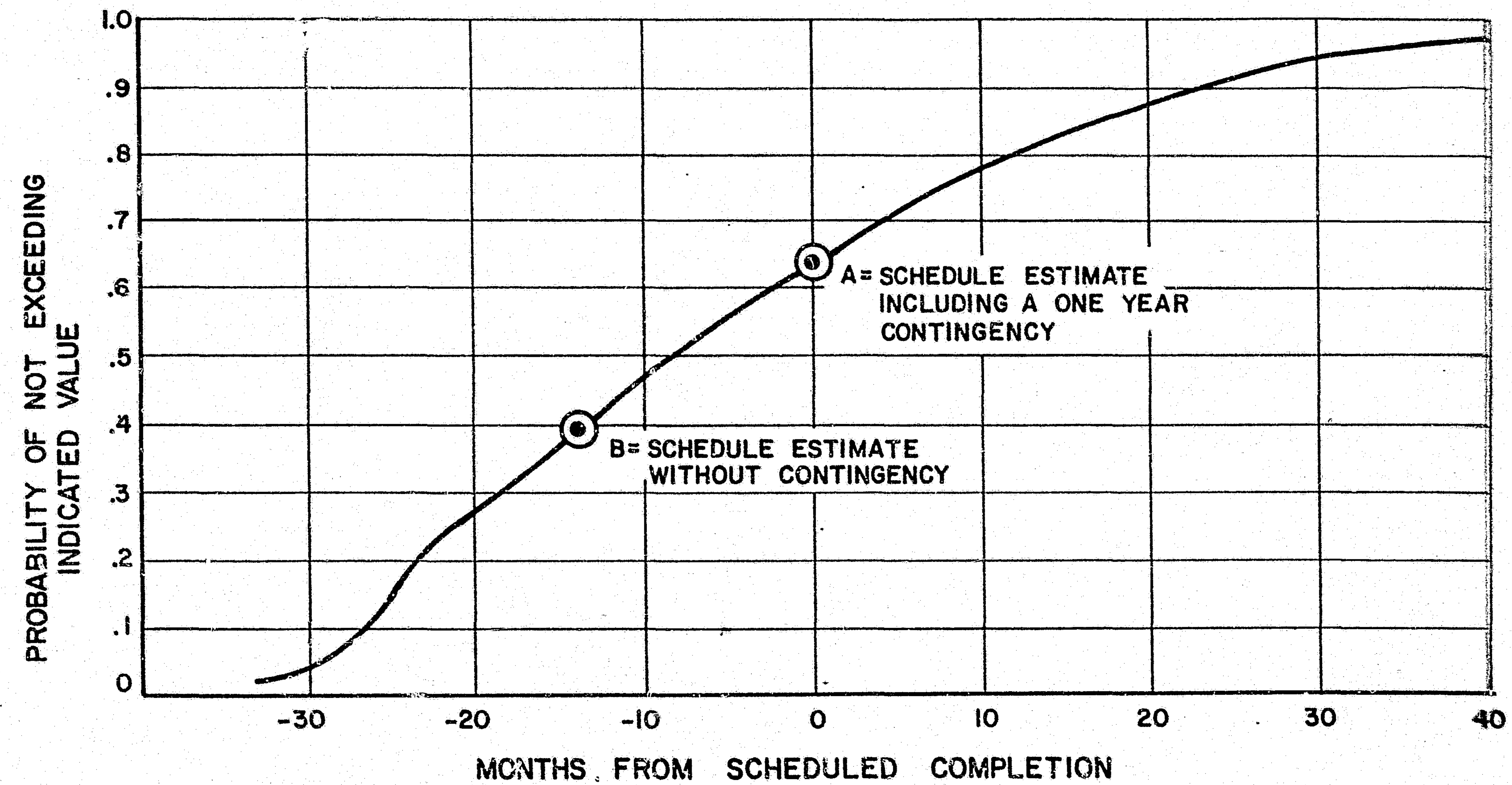
FIGURE 18.9



WATANA SCHEDULE DISTRIBUTION
EXCLUSIVE OF REGULATORY RISKS

ACRES

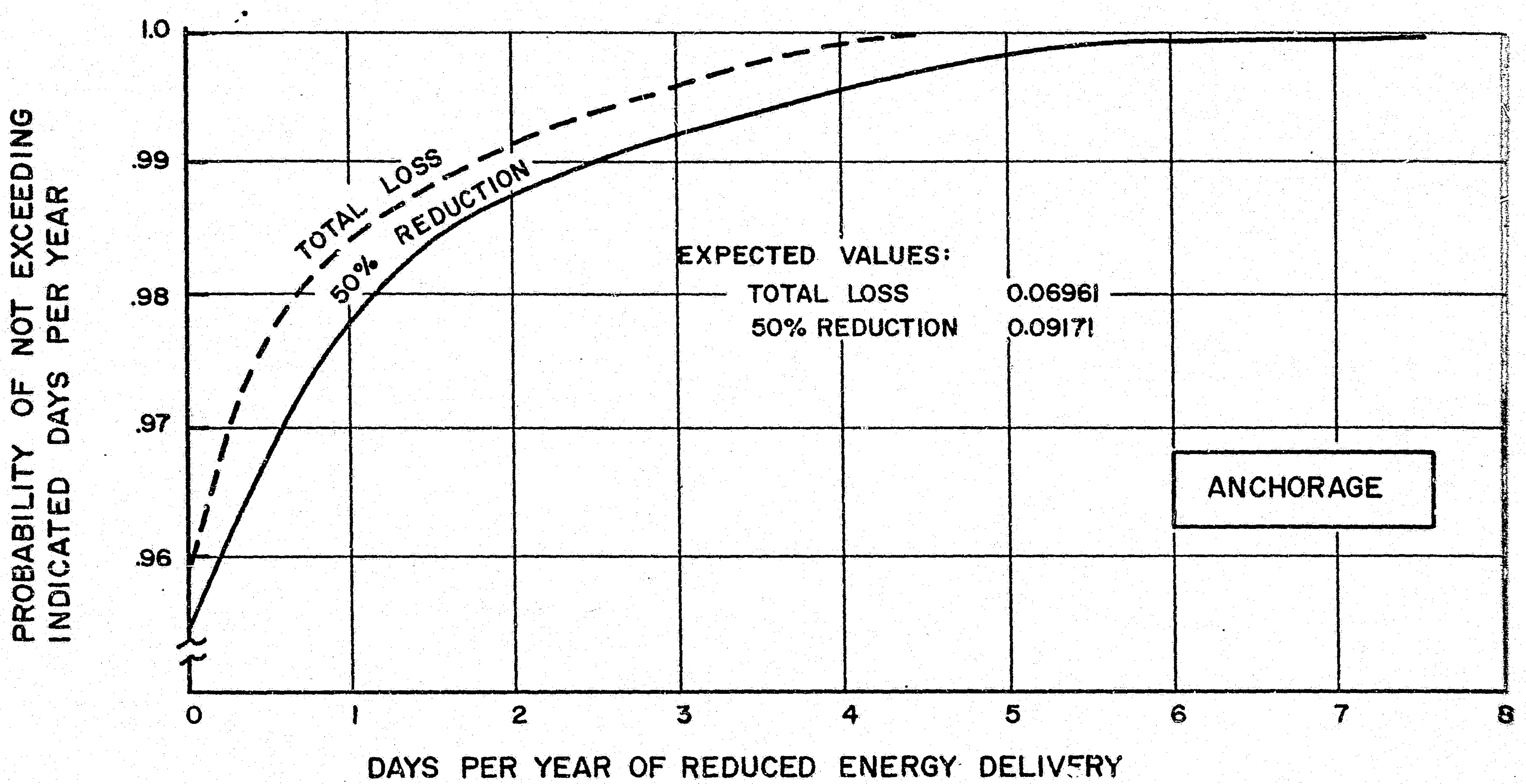
FIGURE 18.10



WATANA SCHEDULE DISTRIBUTION
INCLUDING THE EFFECT OF REGULATORY RISKS

FIGURE 18.11

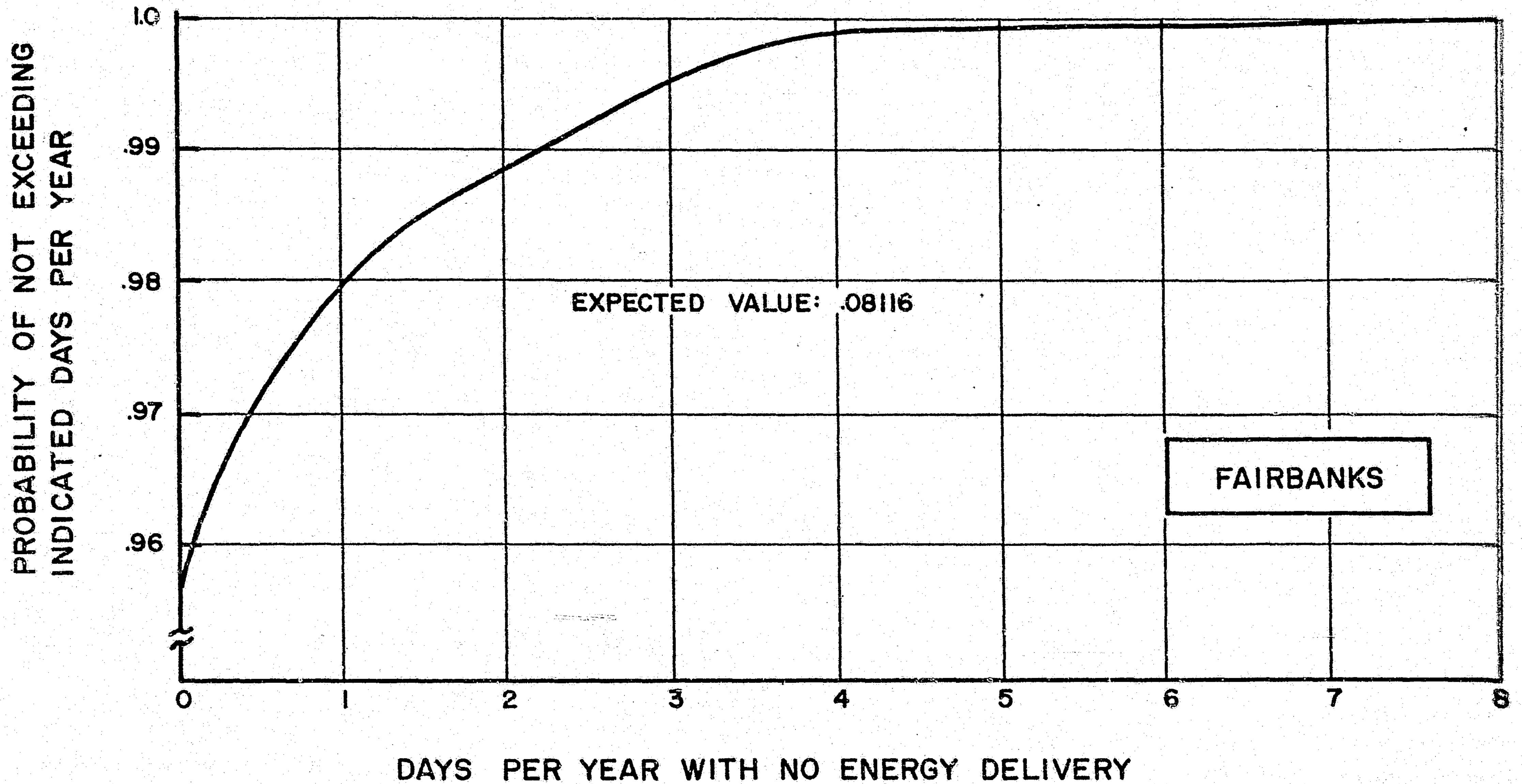
ACRES



CUMULATIVE PROBABILITY DISTRIBUTION FOR DAYS OF REDUCED ENERGY DELIVERY TO ANCHORAGE

ACRES

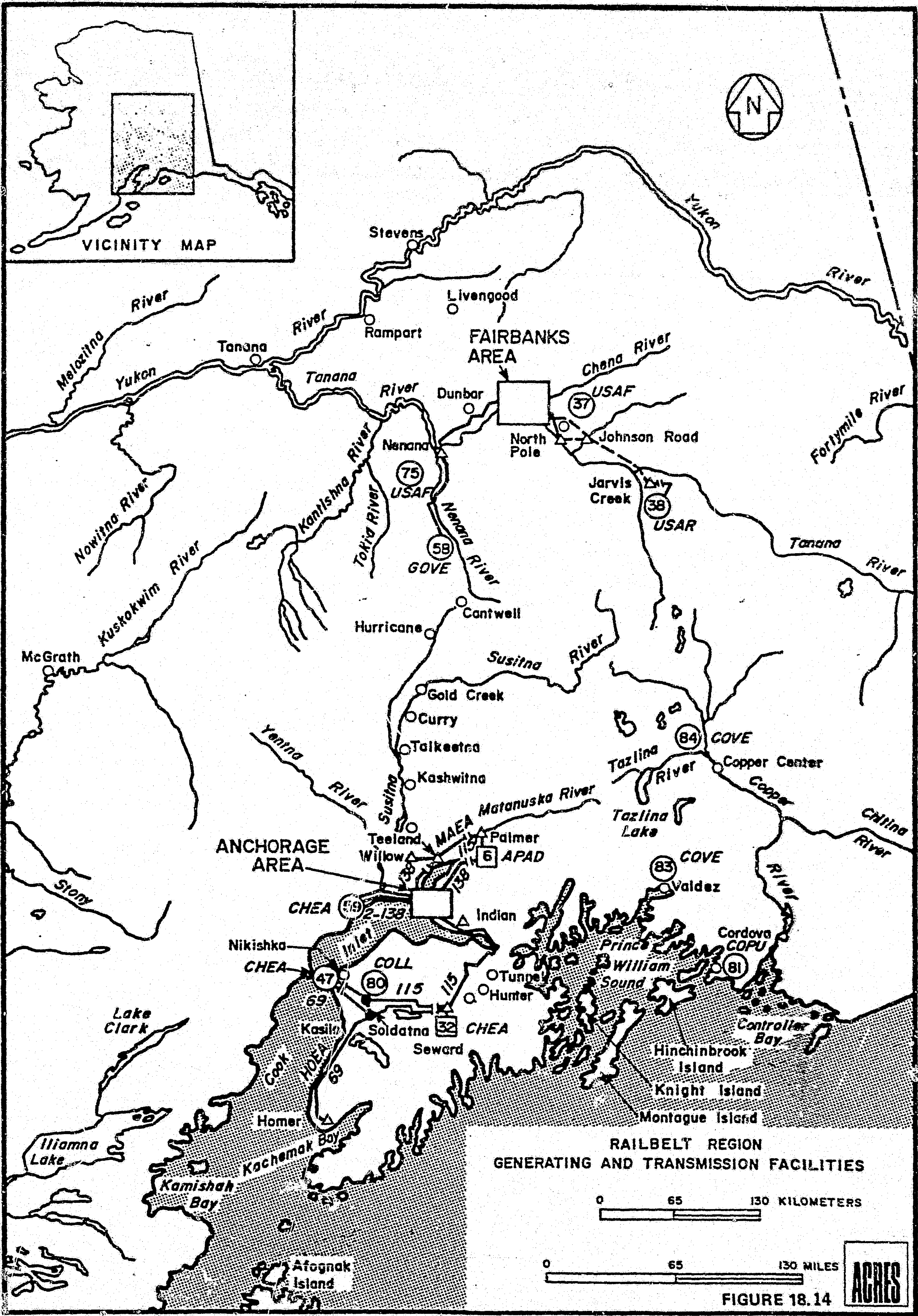
FIGURE 18.12



CUMULATIVE PROBABILITY DISTRIBUTION
FOR DAYS PER YEAR WITH NO SUSITNA
ENERGY DELIVERY TO FAIRBANKS

FIGURE 18.13

ACRES



RAILBELT REGION
GENERATING AND TRANSMISSION FACILITIES

FIGURE 18.14

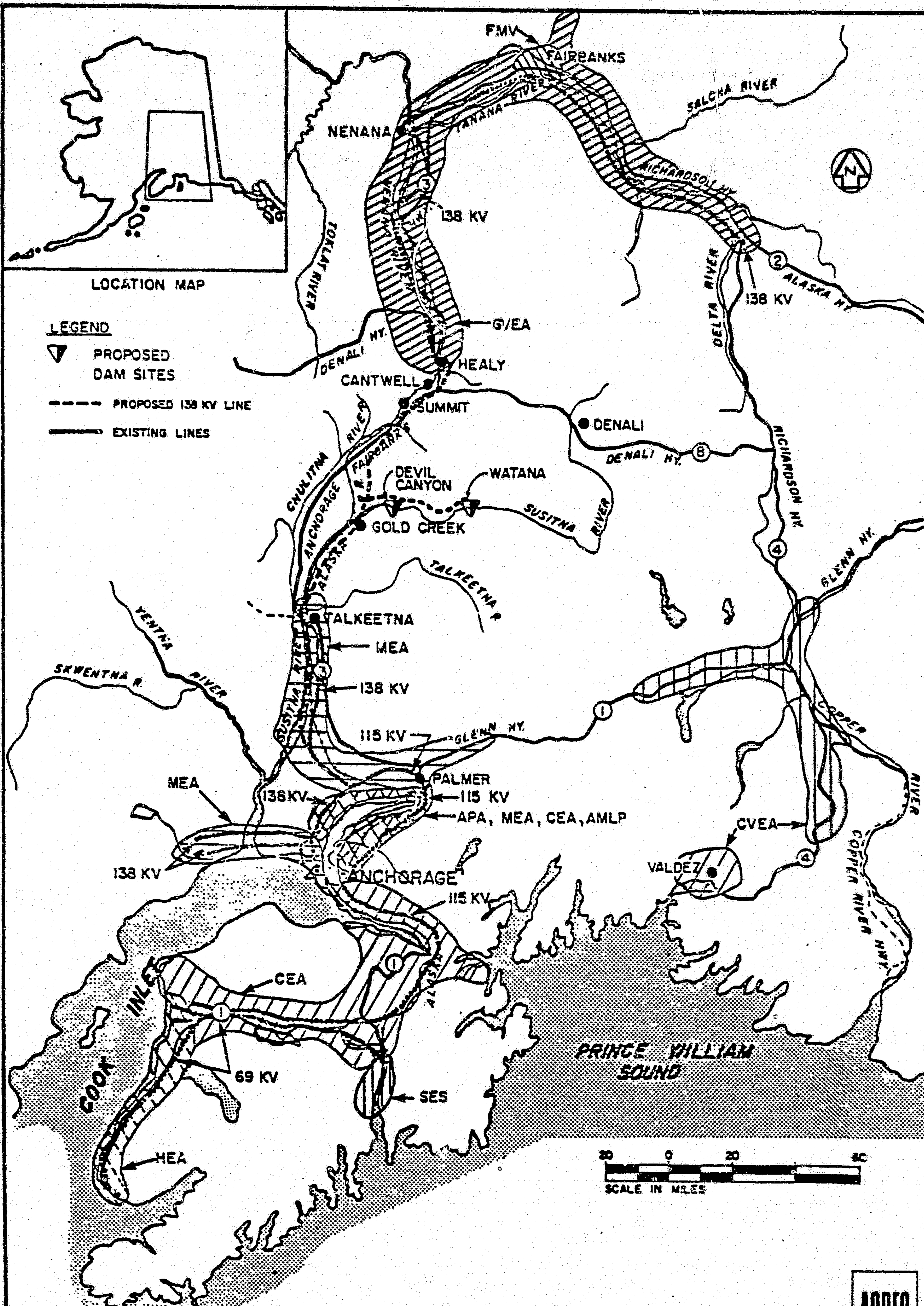
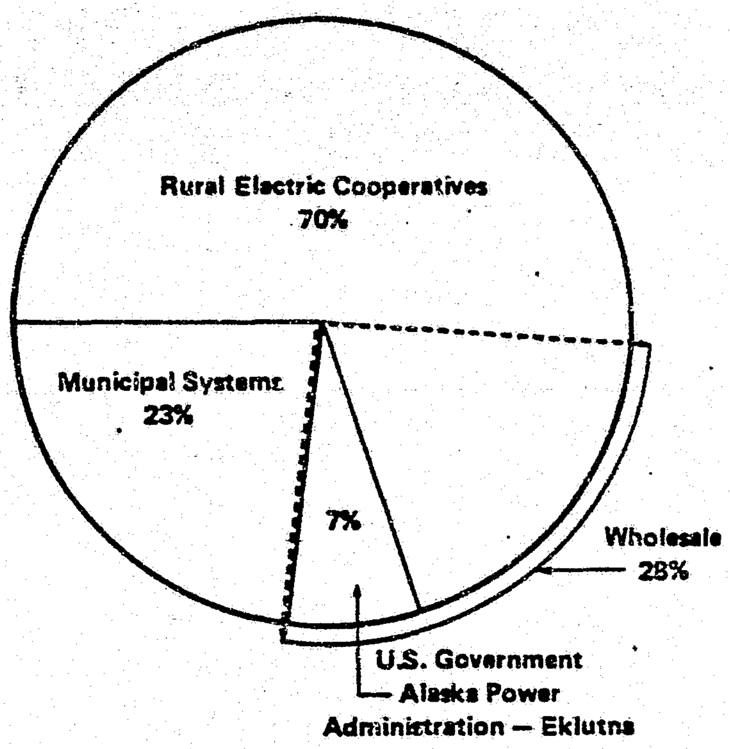
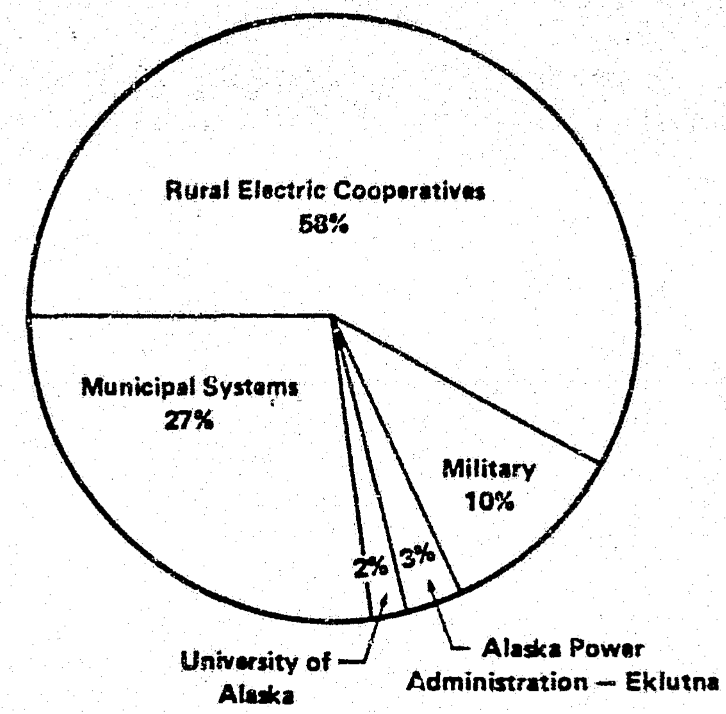

LOCATION MAP
LEGEND
PROPOSED DAM SITES
PROPOSED 138 KV LINE
EXISTING LINES
FMV
FAIRBANKS
SALCHA RIVER
TANANA RIVER
NENANA
RICHARDSON HY.
138 KV
TOKLAT RIVER
DELTA RIVER
ALASKA HY.
G/EA
DENALI HY.
HEALY
CANTWELL
SUMMIT
DENALI
CHULITNA RIVER
DEVIL CANYON
WATANA
SUSITNA RIVER
GOLD CREEK
TALKEETNA R.
YENTNA RIVER
SKWENTNA R.
TALKEETNA
MEA
115 KV
GLENN HY.
PALMER
APA, MEA, CEA, AMLP
CVEA
VALDEZ
COPPER RIVER
ANCHORAGE
CEA
COOK INLET
69 KV
SES
HEA
PRINCE WILLIAM SOUND
SCALE IN MILES
20
0
20
60

FIGURE 18.15

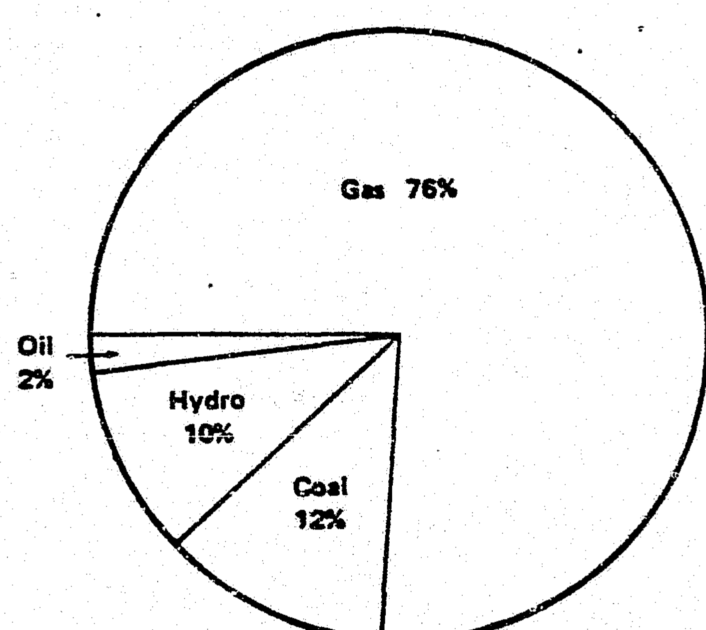


1. Does Not Include Self Supplied Energy from Military Installations and The University of Alaska

A ENERGY SUPPLY
(Based on Net Generation 1980)

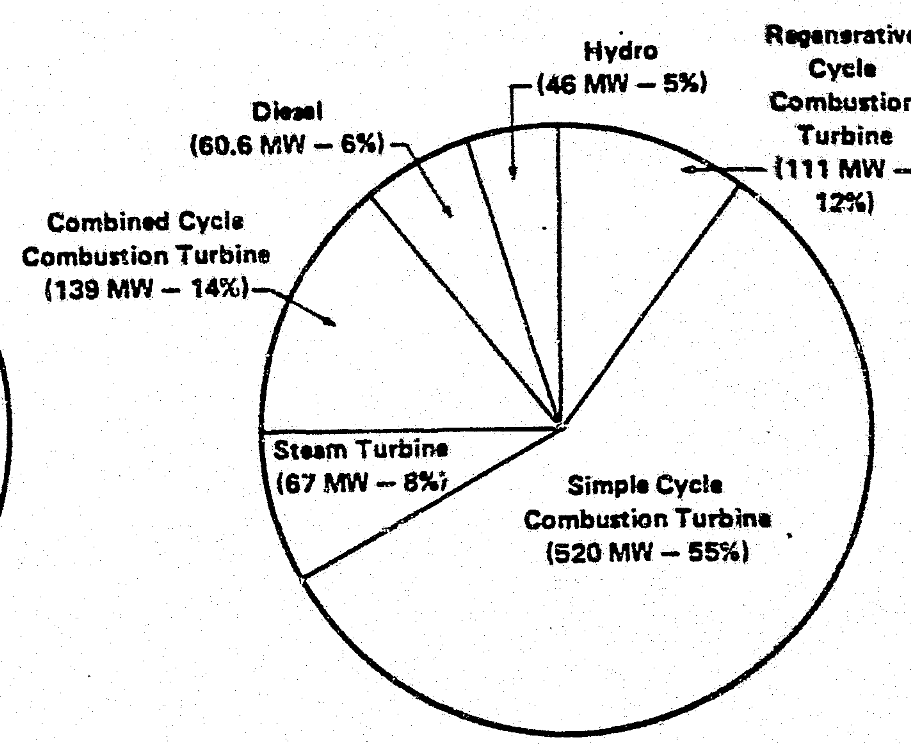


B GENERATING FACILITIES
(Based on Nameplate Generating Capacity 1980)



1. Does Not Include Generation by Military Installations and The University of Alaska

C NET GENERATION BY TYPES OF FUEL
(Based on Net Generation 1980)



D RELATIVE MIX OF ELECTRICAL GENERATING TECHNOLOGY -- RAILBELT UTILITIES — 1980

FIGURE 18.16

ACRES

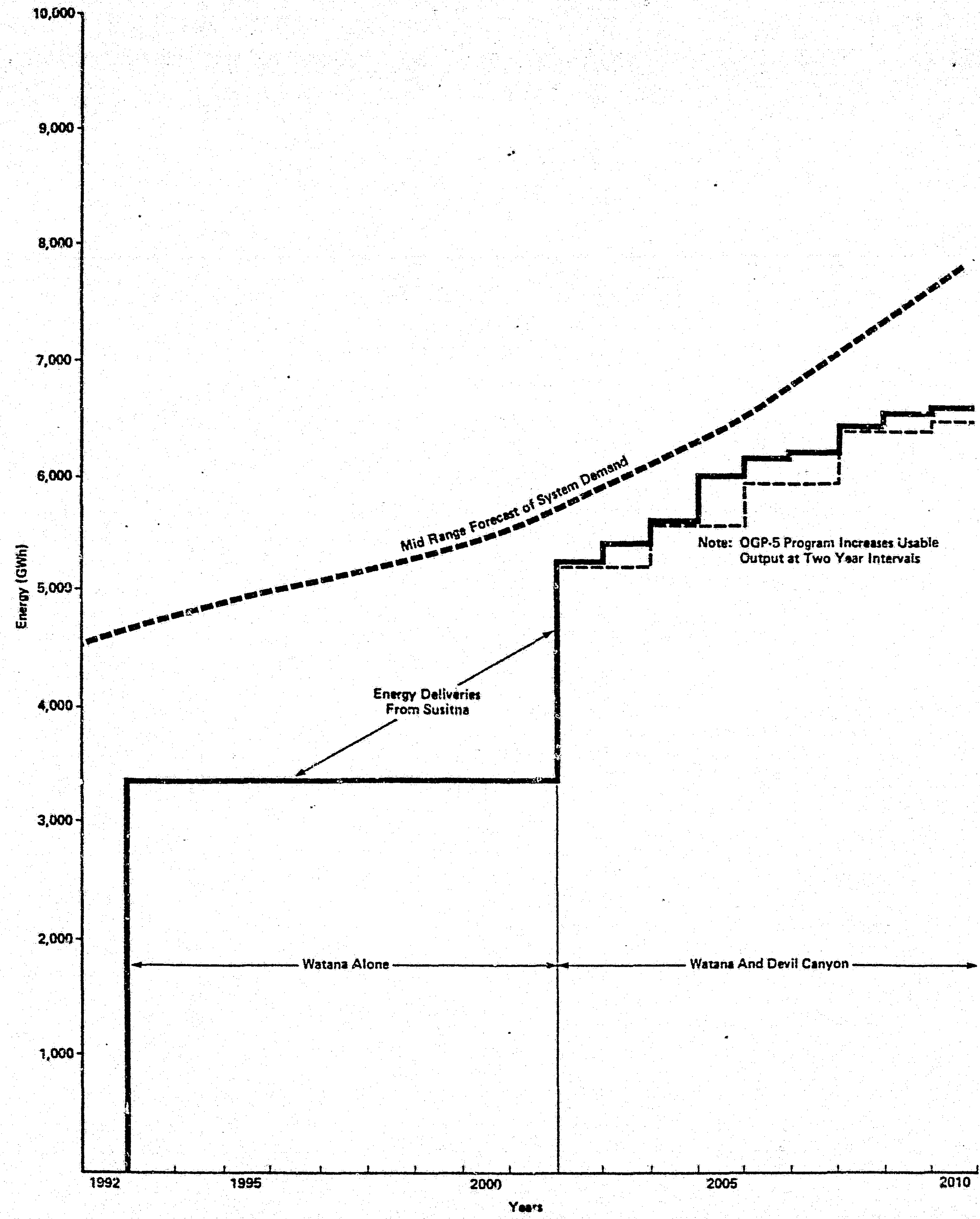


FIGURE 18.17 — ENERGY DEMAND AND DELIVERIES FROM SUSITNA

ACRES

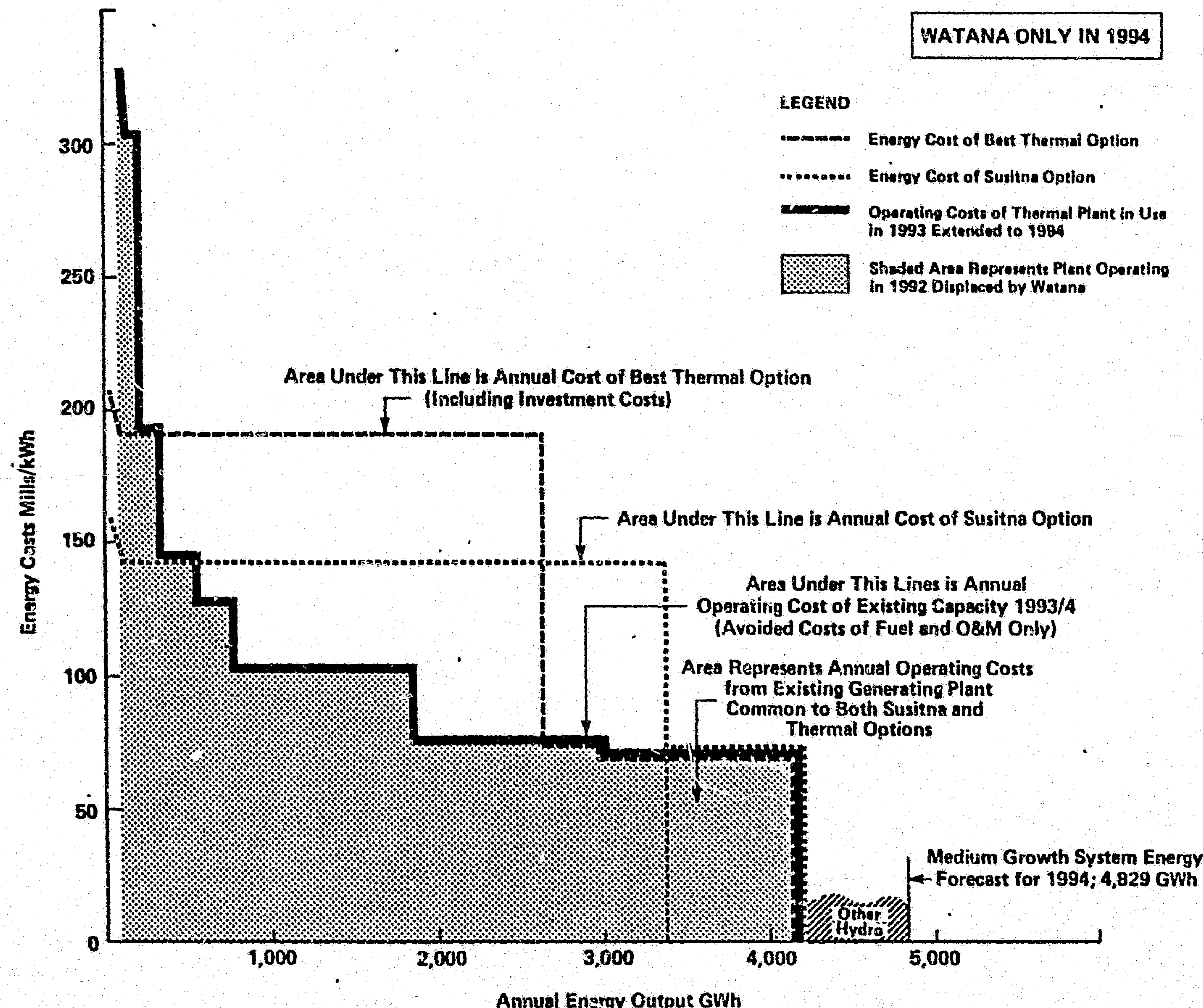


FIGURE 18.18 – ENERGY PRICING COMPARISONS – 1994

ACRES

Rev. 1

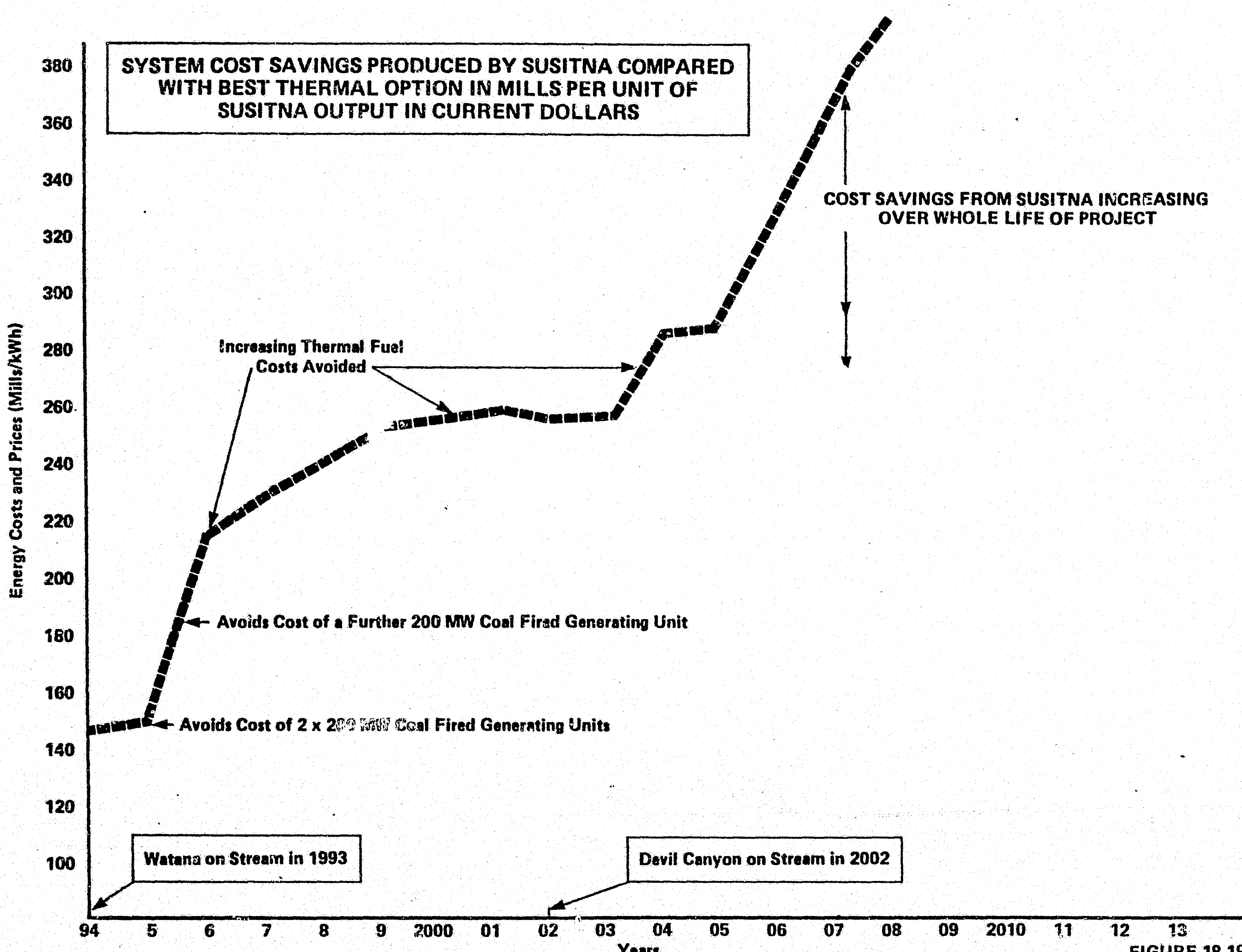


FIGURE 18.19

ACRES

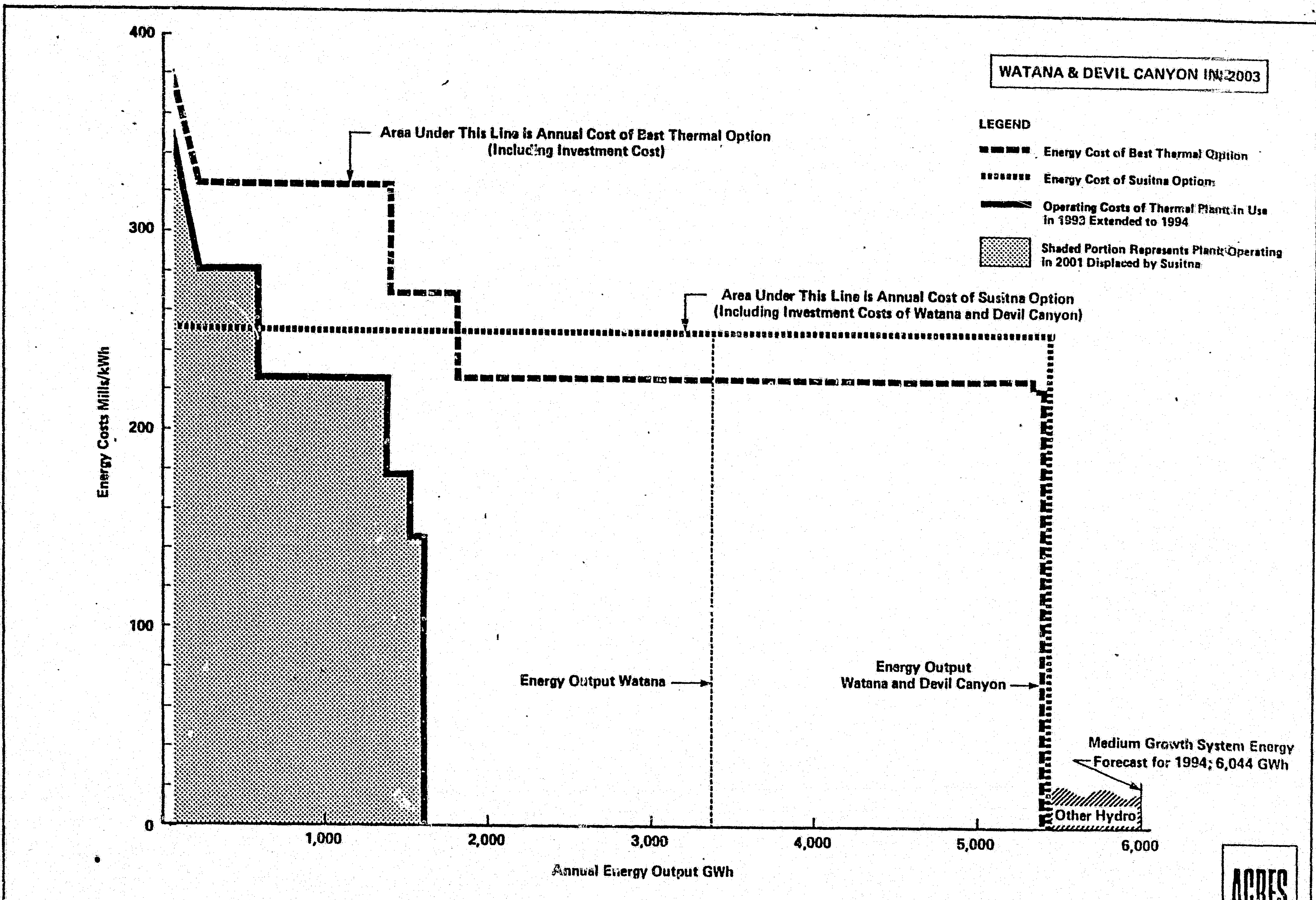


FIGURE 18.20 — ENERGY PRICING COMPARISONS

ACRES

Rev. 1

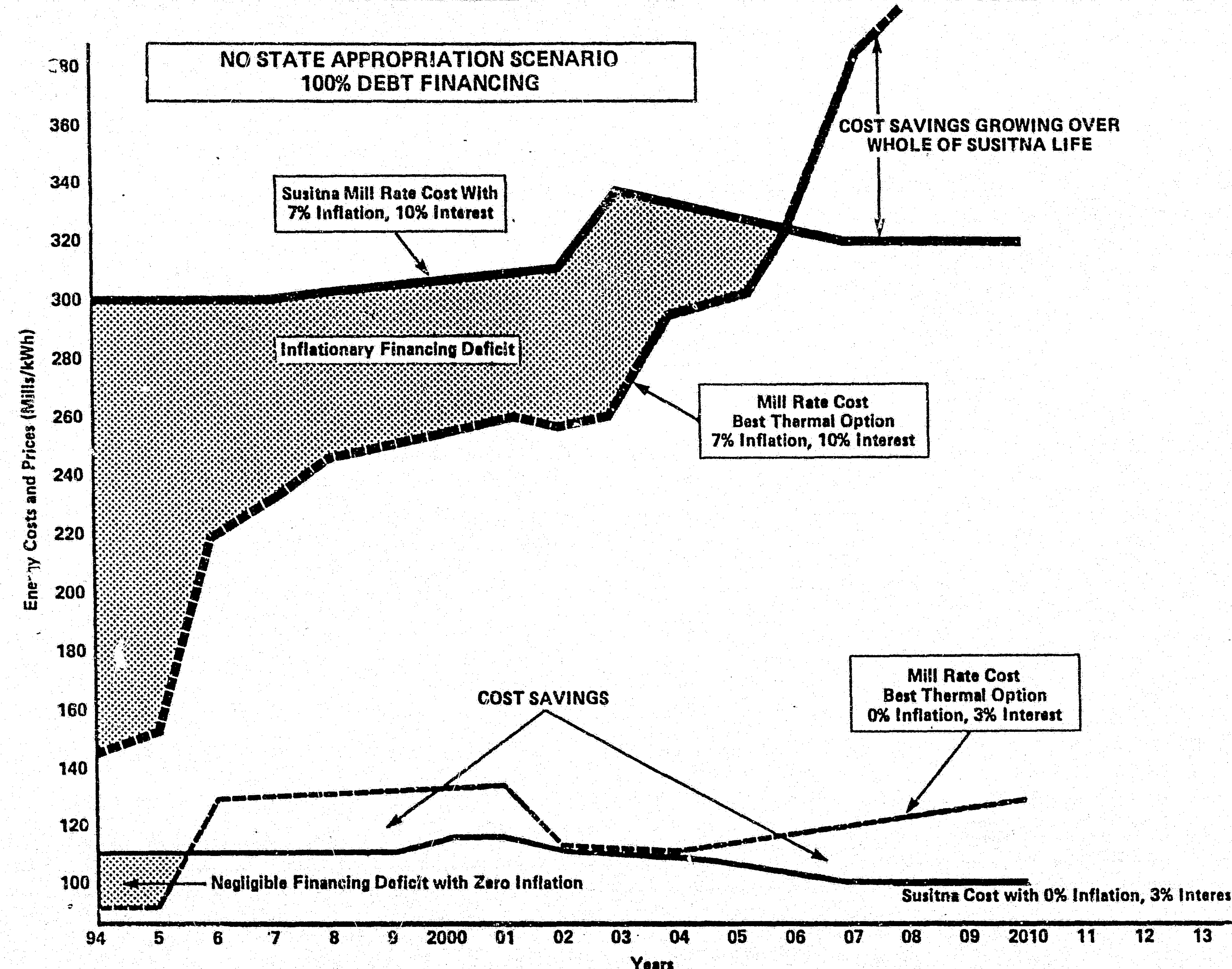


FIGURE 18.21 – ENERGY COST COMPARISON 100% DEBT FINANCING 0 AND 7% INFLATION

ACRES

Rev. 1

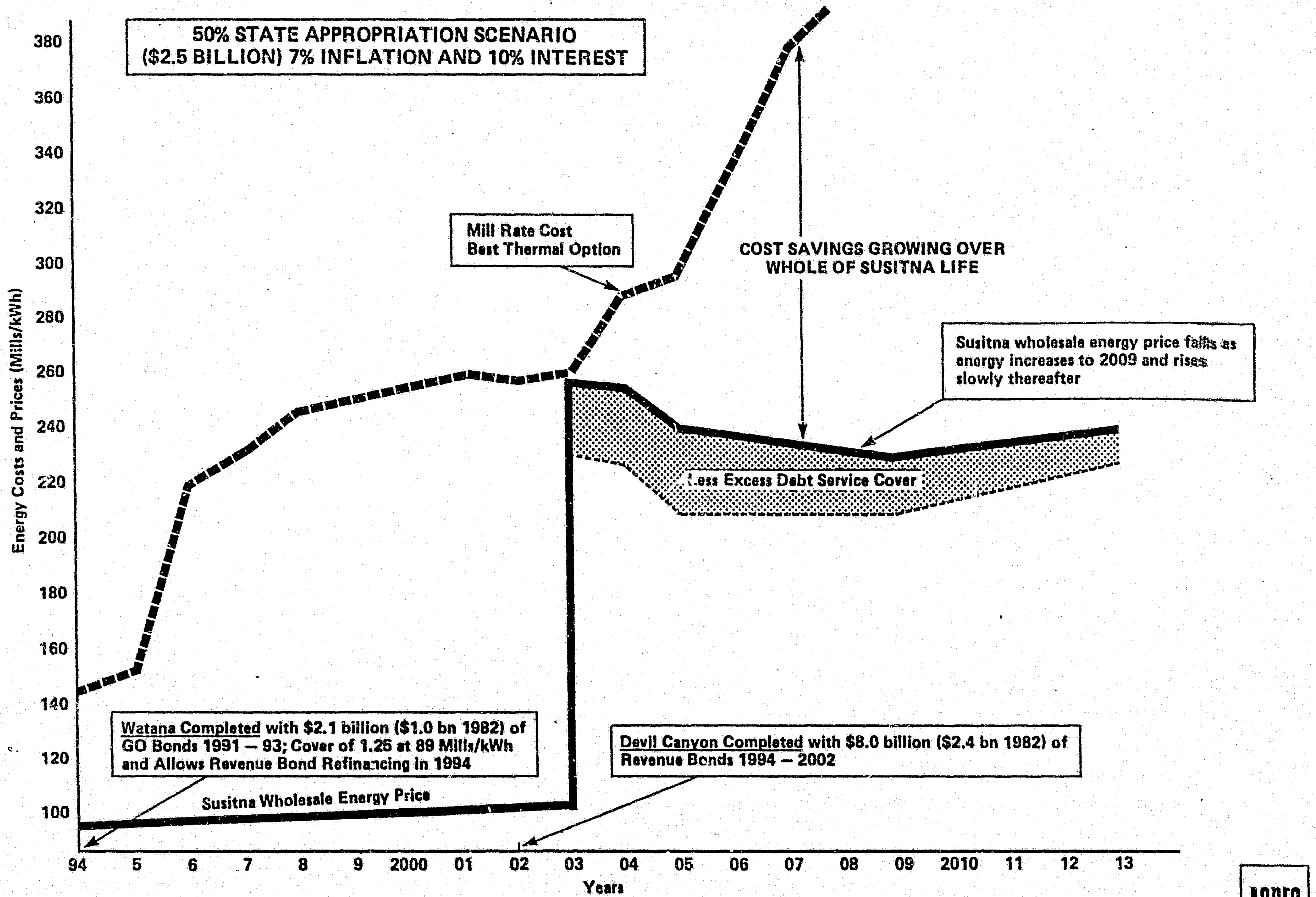


FIGURE 18.22 — ENERGY COST COMPARISON 50% STATE APPROPRIATION SCENARIO

ACRES

Rev. 1

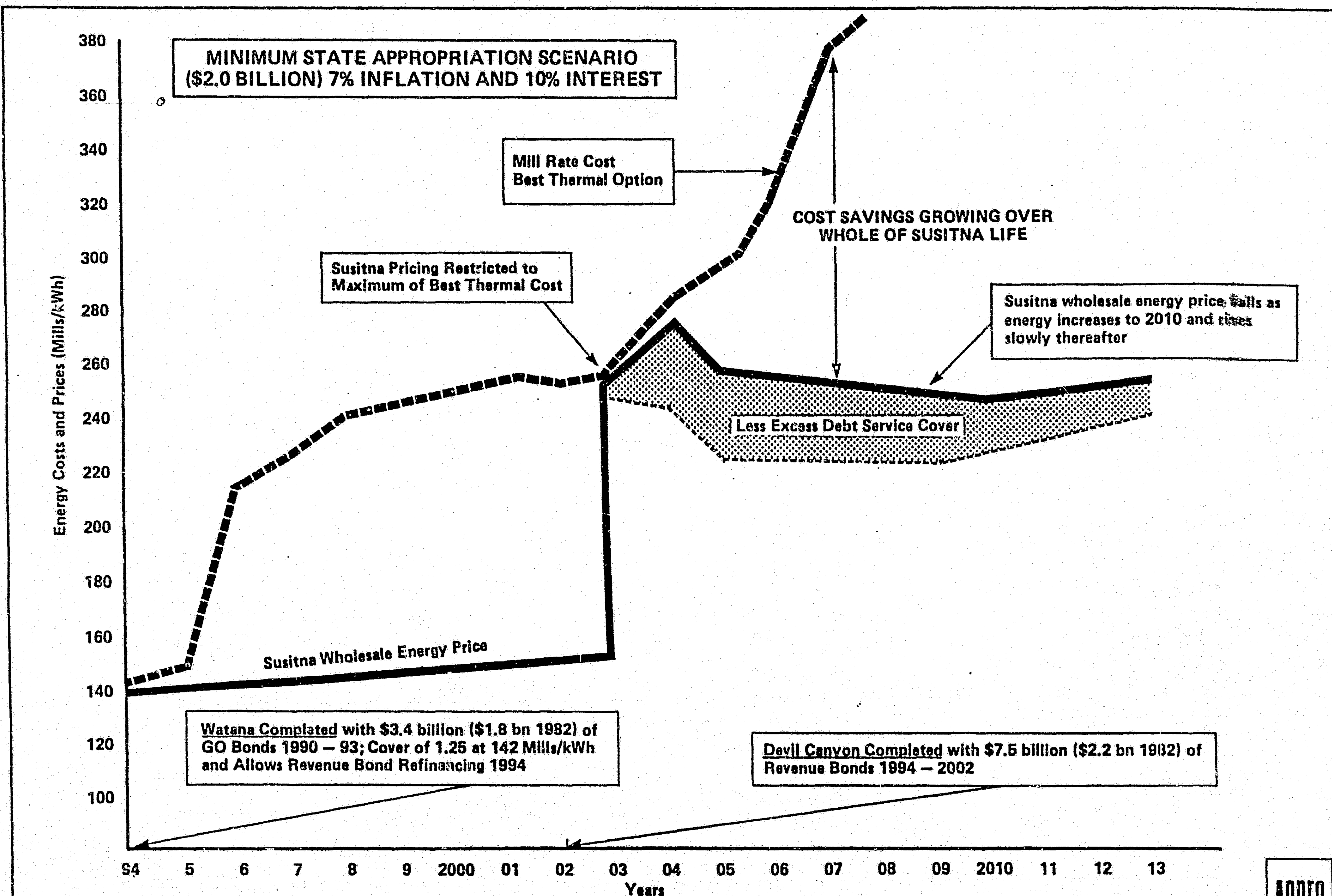


FIGURE 18.23 – ENERGY COST COMPARISON MINIMUM STATE APPROPRIATION

ACRES

Rev. 1

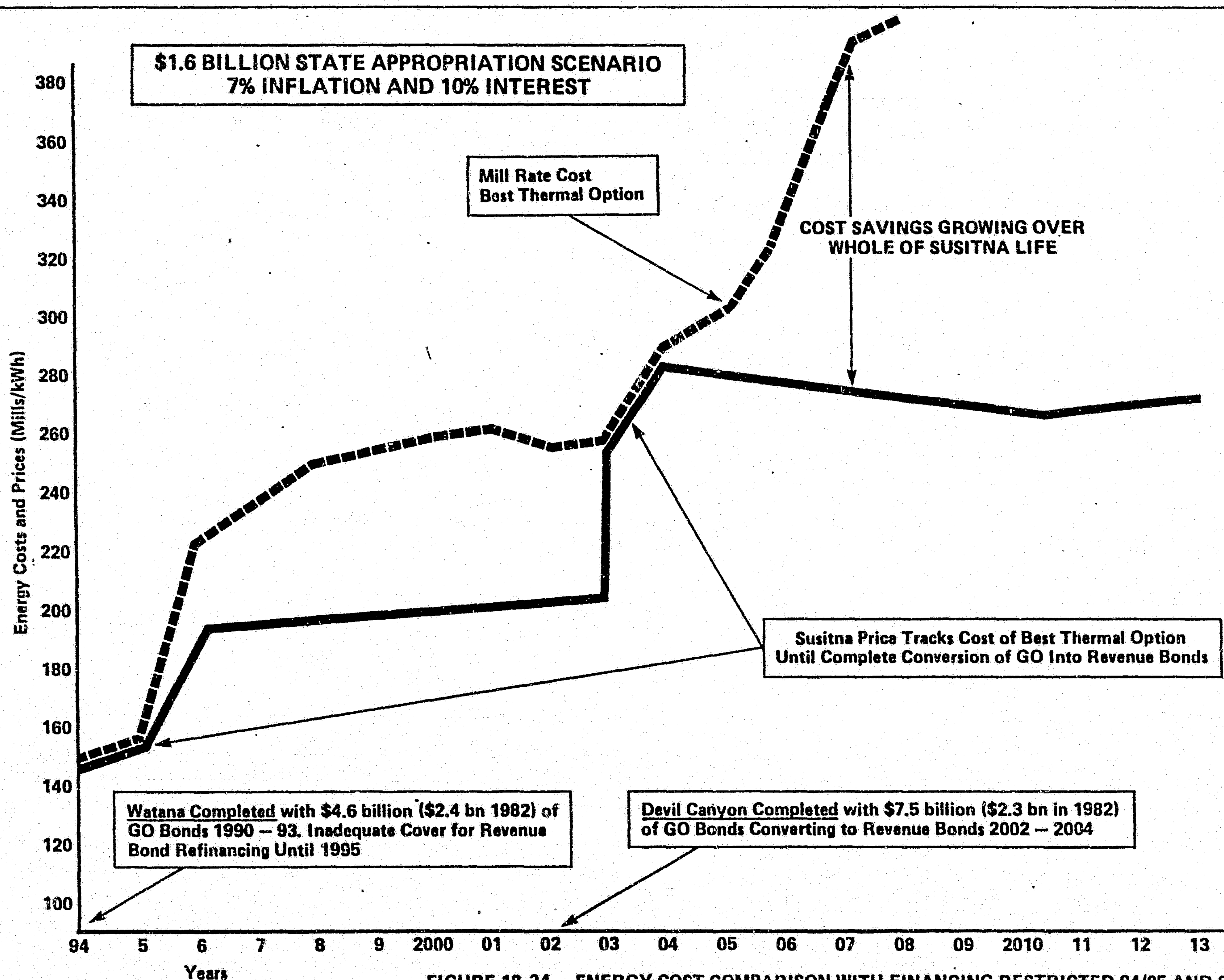


FIGURE 18.24 – ENERGY COST COMPARISON WITH FINANCING RESTRICTED 94/95 AND 03/04

ACRES

SPECIFIC RISK I: RISK OF GO BOND FINANCING OVERRUN

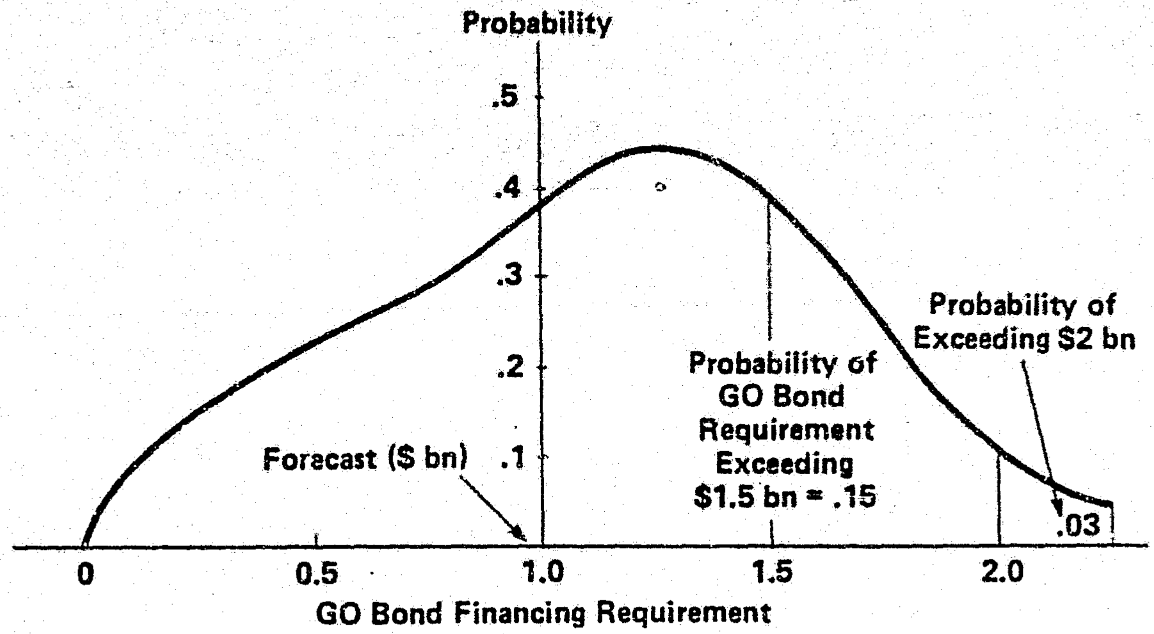


FIGURE 18.25 – GO BOND REQUIREMENTS IN 1982 DOLLARS BILLION

SPECIFIC FINANCING RISK II: EARLY YEAR NONVIABILITY

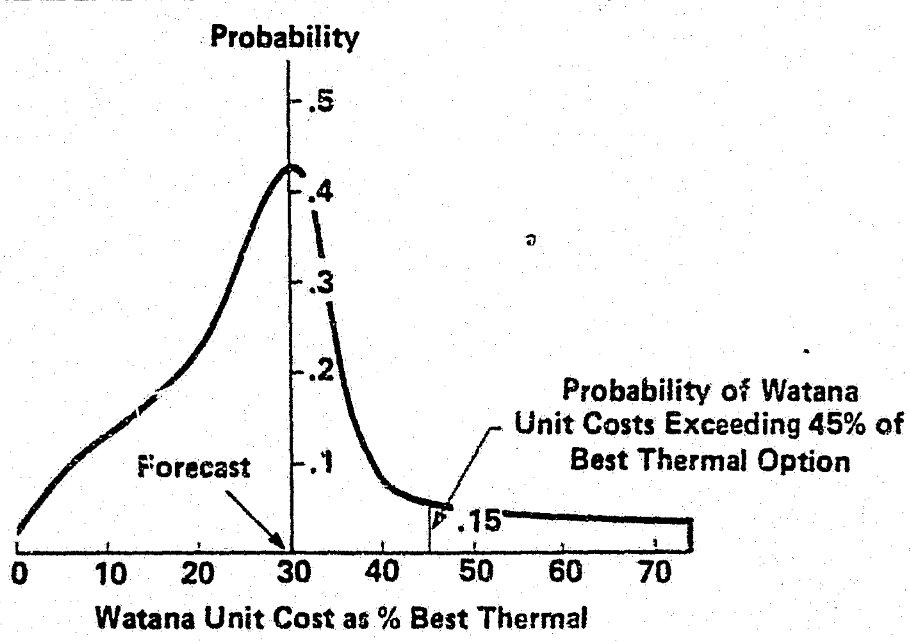


FIGURE 18.26 – WATANA UNIT COSTS AS PERCENT OF BEST THERMAL OPTION IN 1996

AGGREGATE RISK

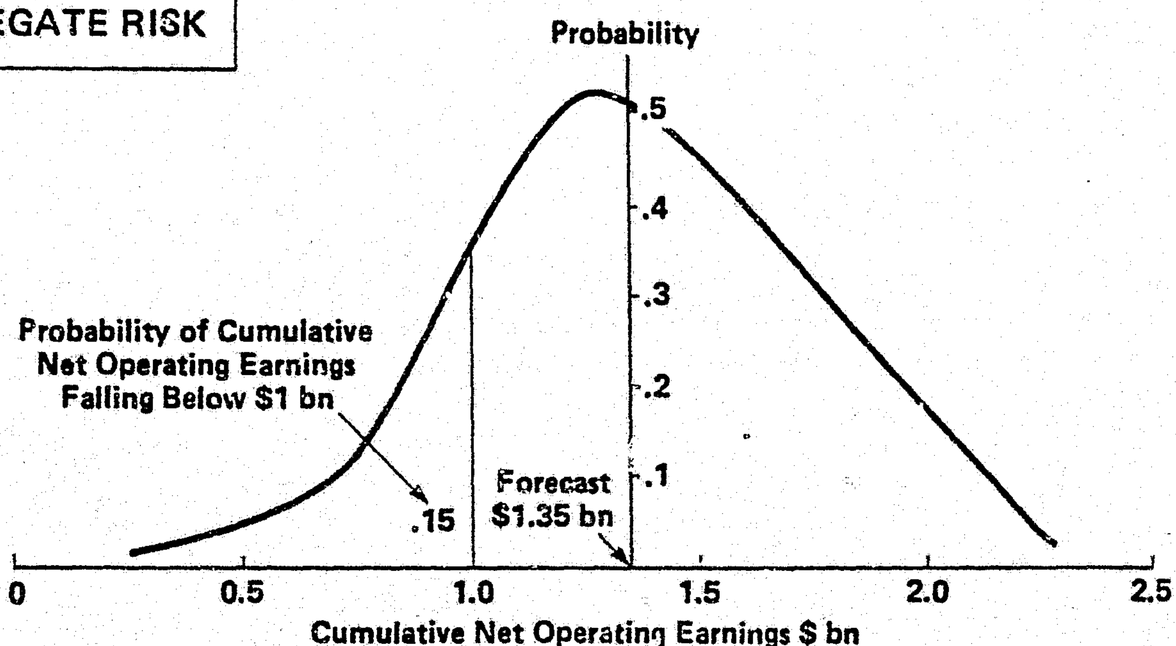


FIGURE 18.27 – CUMULATIVE NET OPERATING EARNINGS BY 2001

ACRES