CHAPTER 5. SITE CONDITIONS

5.1 Field Investigations Performed

5.1.1 Scope and schedule of investigations

The sites of Additional Detailed Investigations were selected based on the preliminary studies described in Chapter 4. Site I-C was selected as the most promising dam site but with an assumption that the water leakage through the karstic limestone located at the dam site can be technically managed. Site I-B was reserved as the site to be taken up when the I-C site would have to be abandoned due to unexpected geological problems in the course of the Study.

The sites to be investigated were determined as follows;

(1) I-C dam site in the Görmel Gorge
(2) The landslide area located on the left abutment of the I-B dam site
(3) Rock quarry and borrow areas around the dam site
(4) Power waterway route from the I-C dam site to the outlet of tailrace tunnel
(5) Underground power house site
(6) Weir site and waterway route of the Erik Diversion Scheme

The Additional Detailed Investigations for these sites were performed succeeding to and integrating the geological investigations which had been ongoing by EİE in the Project site before commencement of the Study. The investigations had been completed by the end of November 1989 except for some geological investigations. The geological investigations for the Study were completed by May 1990 when drilling works of the deep boreholes planned on the right bank of the dam site and power station site were accomplished.

5.1.2 Topographic survey

Topographic survey and mapping were conducted between the middle of June and the end of November 1989 by the Topographic Division of the Project Department of EİE, to prepare necessary maps in addition to those prepared by EİE preceding the Study. These maps are as listed below:

Location	Scale	Surveyed Area (ha)	Nos. of Sheets
Maps prepared before the Study			
Project area	1/5,000	-	42
Dam site I-B	1/1,000	-	5
Maps prepared under the Study			
Dam site I-C	1/1,000	115	4
Outdoor switchyard and tunnel portal	1/500	21.25	3
Outlet of tailrace tunnel	1/500	21.00	3
Erik diversion weir	1/500	6.75	1
Total		164.00	58

A river cross section survey was also carried out by EİE at 4 sections around I-B and I-C dam sites (Fig. 5.1).

5.1.3 Geological investigations

Geological maps were prepared for the reservoir area, the whole Project site including dam site I-C, and the Erik intake site (Plates G1, G2, G3, G22).

The core boring investigation of 7,438.75 m in total depth (5,111.05 m at the pre-feasibility study stage and 2,327.70 m during the Study) was performed as shown in Table 5.2. The borehole location is shown in Plate G2. Lugeon tests, 1,191 times in total (968 times at the pre-feasibility study stage and 223 times during the Study) were performed in these boreholes.

At the pre-feasibility study stage, seismic exploration along 31 profiles and 193 points of vertical electric sounding were performed by EİE, in I-B dam site area (1987), and in I-C dam site and headrace tunnel route (1988). The results are presented in EİE reports. During the Study, seismic exploration by refraction method was performed for 9 lines, 9,360 m in total line length. The locations of these points and seismic lines are shown in Plate G2. The results of analysis are shown in the geological profiles of each seismic exploration line.

A test adit of 42 m in length was excavated in the I-B dam site at the pre-feasibility study stage. Another adit of 231.2 m in length was excavated in I-C dam site during the Study (Plate G2).

Test samples were taken from the boring cores of SK-302, 307 and 313, to test strength of the rock at I-C dam site. The test results are given in Table 5.5.

The Geology and Drilling Department of EİE had prepared a study report of seismicity, which was partly revised by the JICA Study Team.

A micropaleontological and mineralogical study was performed at the pre-feasibility study stage for 53 samples taken in the field in order to confirm the geological age of each limestone block in the study area. Under the Study, 4 samples were tested mineralogically to examine swelling tendency of the rock.

5.1.4 Material tests

The field investigation and laboratory tests were carried out to identify locations, engineering properties and available quantity of construction materials.

28 test pits were excavated at the pre-feasibility stage and 16 test pits under the Study, to take samples for laboratory tests. The depth and numbers of sampling are listed in Table 5.2. Location of the test pits are shown in Plate G27. The test samples were taken from the test pits and boring cores. The test items, quantity and results are summarized in Table 5.3.

Two boreholes of SK-311 and 312, 109 m in total length, were drilled in the proposed quarry site in order to test rock quality and to estimate available quantity. The location of boreholes are shown in Plate G28.

5.1.5 Other surveys

(1) Hydrological measurements

Hydrological Survey Department of EİE conducted flow measurements at 4 stations around the dam sites between March 1989 and August 1990.

(2) Transmission line route survey

The proposed transmission line route for the Project was preliminarily reconnoitered in November 1989 by running with jeep on motorable roads along the conceivable routes between Ermenek and Seydişehir.

(3) Compensation survey

A compensation survey for the Project was carried out by EİE on the legal basis of the following laws and manual:

(A) Law Number 6200: Establishment of the DSİ

DSİ was established in 1953 with the responsibility to implement irrigation, energy and urban water infrastructure projects. Under Article 2 of Law Number 6200, the authority to purchase and/or to expropriate privately-owned land for project implementation purposes has been given to DSİ.

(B) Law Number 6830: Expropriation Law

This law establishes the procedures for transferring the ownership of privately-owned "unmovables" or land to state-ownership and use.

(C) DSİ's manual

The compensation survey was carried out in accordance with the "Manual for Expropriation Studies, 1984", prepared by the Planning and Analysis Division of DSİ.

The compensation survey for the proposed reservoir area was conducted by EİE first in September 1988 for the elevation range up to 650 m. At the Additional Detailed Investigation stage between August and September 1989, the survey was supplemented by EİE for the elevation range between 650 and 750 m, since it was found through the preliminary study that a prospective HWL would exceed 650 m. The survey covered 7,367 ha in total between 510 m and 750 m in elevation.

The survey results were reviewed by the Study Team.

(4) Environmental impact study

An environmental impact study has been based primarily on existing data and study reports and limited field investigation. The field investigation was conducted during October 30 through November 3, 1989. Additional data were obtained from regional/district offices in Ermenek. Consultation was made also to scholars at Haçetepe University to clarify particularly ecological aspects of the environment.

(5) Construction works survey

Information about the power source and communications system available for the construction works of the Project was collected. Data including prevailing prices of materials, equipment and labors were collected and unit construction prices were analyzed. Outlines and construction records of similar projects were also investigated. The existing conditions of the road network around the Project area were investigated in November 1989.

5.2 The Ermenek Basin

The Ermenek river rises from the Toros mountains at about 2,500 m in elevation and takes the streams such as Gökdere, Kücüksu, Zeyve and Erik. It flows firstly towards the south and then towards the east and finally joins the Göksu river near Mut town at about 100 m in elevation (Plate P1).

The catchment area of the Ermenek river basin is about 3,621 km^2 at the confluence of the Göksu and Ermenek rivers. The surface boundary of the basin is not clear especially on its southern boundary on the flat and wide plateau made of limestone. The highest elevation of the basin is 2,877 m. The riverbed elevation is about 500 m at the Ermenek dam site, and the average elevation of the basin upstream of the dam site is about 1,600 m.

About 50 per cent of the basin is covered mainly by juniper type of trees. There are no significant diversions of water from the Ermenek river.

5.3 Geology

5.3.1 Regional topography

The Göksu river basin is located in the middle part of the Central Taurus Mountain Range (Central Taurides), which is developed along the Mediterranean coast. The Central Taurus Mountain Range is a geographical subdivision of the Taurus Belt in the south-eastern Anatolia.

The Taurus Belt is divided into three ranges; Western, Central and Eastern Taurus Mountain Ranges. These ranges are defined as the areas by two major tectonic structures. In the west of Alanya city, NW-SE trend structure exists between the Western and Central Taurus Mountain Ranges, and in the east near Adana city, NE-SW structure (Eçemis fault) is the border between Central and Eastern Taurus Ranges (Refer to Fig. 5.3).

The Project site is located in the middle reaches of the Ermenek river. The proposed dam sites are located to the south of Ermenek city, near the Görmel bridge. The power house area is located about 10 km downstream of the dam site. The Erik intake site is located about 4 km upstream of the confluence of the Ermenek and Erik rivers.

The topography in the surrounding area of the Ermenek river basin presents rather flat plateau of 1,500-2,200 m in elevation, which is formed by mainly thick and horizontally bedded limestone layers of the Tertiary Miocene Ermenek Formation.

In the area between Küçüksu river and the Zeyve river which forms the main part of the proposed reservoir, both the banks have rather gentle slopes; the foundation rock is mainly marl of the Tertiary Görmel Formation; some remnants of old landslide are seen on the slopes of both banks, but mainly on the left bank. These are now in stable condition, and no sliding activity is seen in this area.

In the area upstream of the Küçüksu river and downstream of the Zeyve river, both banks have steep mountainous slopes, where the rocks are mainly composed of the Cretaceous Ermenek Ophiolitic Melange. At those places where the river flows in limestone blocks, both the banks show very narrow and high gorges.

Most of the flows of the Ermenek river originate from the Kapiz spring and the Nadire spring located to the west of the Project site. The uppermost reaches are located on the limestone plateau. The flows of the tributaries also originate from springs, such as the Balkusan spring, Zeyve spring, Erik spring and so forth. The river flows upstream of these springs are generally low or almost nil, especially in the dry season. These springs are seen on the limestone blocks of the Ermenek Ophiolitic Melange and on the foot slopes of steep cliffs of Miocene limestones, which are distributed on the top of plateau on both the basin boundaries.

5.3.2 Regional geology

The geological formations distributed in the reservoir area are summarized in Table 5.1 and Plate G1.

The Upper Cretaceous Ermenek Ophiolitic Melange outcrops in the wide areas around the Project site as a basement. It is composed of Carboniferous-Upper Cretaceous blocks of sedimentary rocks, mainly limestone, in different character, and matrix layers. The matrix layers are formed by diabase, serpentinized peridotite, gabbro, graywacke, graywackish sandstone, schist, conglomerate and so forth.

The Cenozoic Tertiary formations, namely the Görmel Formation of Lower Miocene age and the Ermenek Formation of Middle Miocene age are predominant in the area.

The Görmel Formation is composed of mainly marl, and partly claystone, sandstone, clayey to sandy limestone and conglomerate. Some coal bands exist in the Görmel Formation. A coal mine is located on the right bank of Küçüksu river to the southwest of Ermenek city.

The Görmel Formation is in angular unconformity with the lower formation of the Ermenek Ophiolitic Melange, and this is overlain by the Ermenek Formation with angular unconformity.

The Ermenek Formation outcrops at rather high elevations, in the northern and southern parts of the study area, with cliffs along the Ermenek valley, and is composed of mainly limestone, partly sandy limestone, sandstone and marl. The Ermenek Formation overlies Aladağ group, the Ermenek Ophiolitic Melange and the Görmel Formation with angular unconformities.

The Quaternary deposits, such as terrace deposits, talus deposits and so forth are seen widely on the slopes and along the rivers.

The most recent and main tectonic deformation was developed during the period of Palaeogene to Miocene in Tertiary age. After that, the area was uplifted by the effect of epeirogenesis and then the various erosion occurred during late Tertiary and early Quaternary age, and the present topography of the region was formed.

Several faults are seen on the Nadire, Azitepe and Kükürce limestone blocks of the Melange. However, most of these are geologically minor faults. No large fault structures were recognized in the Görmel and Ermenek Formations.

Thrust fault is seen in the west part of the study area near Nadire where the Aladağ unit overthrusts the Ermenek Ophiolitic Melange. The age of this thrusting is estimated to be Palaeogene of Tertiary age.

5.3.3 Site geology

(1) Dam site I-B

(A) Geological conditions

The geological profile of the alternative dam site I-B is shown in Plate G4. The left bank above 550 m in elevation is formed by very thick debris (partly 50 m or more) of landslide. The dam axis of site I-B is located a little upstream of this debris area. The foundation rock is marl of the Tertiary Görmel Formation. The river width is about 600 m. The surface of this section is covered by Quaternary deposits, such as terrace deposits, talus deposits and riverbed deposits, with thickness of 10 to 30 m. The foundation rock is marl. The right abutment is formed by mainly marl, with some intercalation layers of thin sandstone and conglomerate of the Görmel Formation. No major fault structures were found in this dam site.

(B) Rock properties of marl

The foundation rocks are mainly marl in the whole section of the dam, which is typical soft rocks. Slightly weathered to fresh rocks of marl correspond to CM class (rock classification of K.Kikuchi Et.al., refer to Table 5.4).

The rock properties are estimated to be as follows.

Rock classification	: CM (soft rock)
Compressive strength	: 100-200 kg/cm^2
Static modulus of elasticity	: 10,000-15,000 kg/cm^2
Cohesion	: 10-15 kg/cm^2
Internal friction angle	: 30-40 degrees
Static Poisson's ratio	: 0.15-0.2

(C) Permeability

There will be no serious problems about water leakage, because the foundation rocks on this dam axis are all marl, sandstone and conglomerate of the Görmel Formation and of low permeability.

(D) Foundation treatment

Curtain, blanket and consolidation groutings will be required along the dam axis below impervious core zone and along the spillway axis for treatment of permeability and uniting the foundation rocks (see ANNEX-A in Volume 3 for grout design).

(E) Slaking

Marls, even in fresh rocks, have a tendency to slake easily by exposing to air. Therefore, some special care will be required during embankment works of impervious core zone.

(2) Dam site I-C

(A) Geological conditions

There are 3 alternative dam axes, I-Ca, I-Cb and I-Cc in the Görmel Gorge (Plate G3). The geological conditions of each alternative axis are shown in the geological map and profiles (Plates G5, G6 and G7). All the 3 alternative axes are located in an upper Jurassic-Cretaceous limestone block in the Ermenek Ophiolitic Melange. This limestone is hard and massive but has many solution cavities.

Three fault structures, namely F-1, F-2 and F-3 were confirmed in the limestone block through the field reconnaissance (Plate G3). According to the results of borehole SK-310, there may not be fault structures below the riverbed.

The groundwater level in the left bank rises towards the mountain side along the boundary between the limestone and matrix of Melange. In the right bank, it rises slightly towards the mountain side (refer to Plate G13).

The main joint systems measured in each dam site are as follows.

(a) Dam site I-Ca and I-Cb

Ja: NS-10^{O}E/vertical : Interval of 1-3 m.
Continuity is low.
Jb: N70^{O}E-EW/90-70^{O}NE: Interval of 1-3 m.
Continuity is medium.
Jc: N40-80^{O}E/30-40^{O}NW: Interval of 10 m or more.
Continuity is high.

In the test adit:
N70^{O}W/65^{O}N
N30-40^{O}E/20-35^{O}N (Jc)
N35-40^{O}E/60-65^{O}N (Jc)

(b) Dam site I-Cc

Je: N10-30^{O}W/vertical: Intervals of 1-3 m.
Continuity is low.
Jf: N70^{O}E-EW/90-70^{O}NE: Intervals of 1-3 m.
Continuity is high.
Jg: NS-30^{O}E/50-70^{O}NE : Intervals of 10 m or more.
Continuity is high.

(B) Permeability conditions of the right bank

The permeability conditions of limestone in the right bank are shown in Plates G12 and G13. These may be summarized as follows:

- Area from ground surface to about EL. 500 m is highly pervious and karstic limestone.
- Area approximately between ELs. 500 to 400 m is low pervious limestone, but possibly has solution cavities in some places.
- Area below about EL. 400 m is low pervious limestone, with very rare possibility of cavity exist-

ence.

- Area near boreholes SK-316 and 314 is rather high pervious limestone, and cavities are seen for the depth until about EL. 350 m.

(C) Engineering geology for I-C dam site

(a) Foundation rock

The slightly weathered to fresh limestone at this site corresponds to B to CH class of rock classification (K.Kikuchi Et.al.), and will be appropriate for the foundation of a high arch dam (refer to Table 5.5). The necessary excavation depth of foundation rocks was estimated at about 30-40 m for both the abutments near the dam crest elevation; and 5-10 m for the riverbed.

The rock properties in slightly weathered to fresh conditions (CH to B class) are estimated as follows:

Rock classification	: CH to B (hard rock)
Compressive strength	: 700-800 kg/cm^2
Static modulus of elasticity	: 80,000-200,000 kg/cm^2
Cohesion	: 40-50 kg/cm^2
Internal friction angle	: 40-55 degrees
Static Poisson's ratio	: 0.25-0.3

(b) Foundation treatment

Consolidation and curtain groutings will be required for the dam foundation to unite the foundation rocks and to improve the permeability. Consolidation grouting will be required for the whole dam foundation area, with hole intervals of 2 to 4 m and depth of 10 to 20 m.

(c) Grout curtain

The right and left ends of the grout curtain can be connected to the marl layer of the Görmel Formation. In the left bank, the limestone becomes thinner towards upstream. Since the rocks below the limestone are low pervious ophiolitic, the bottom of the grout curtain can be raised along the bottom of the limestone. The grout curtain should horizontally be connected to the ophiolitic rocks (Plate G12).

(d) Treatment of fault F-2

At I-Cc dam site, treatment of Fault F-2, which is located in the right abutment, will be required down to an elevation of about 400 m.

(3) Power house

The geological conditions of the power house area are shown in Plates G15 and G16. The results of borehole SK-102 are as follows: (a) The sections from the surface to 156 m in depth (EL. 469 m), and between 263.2 m (EL. 361.8 m) and 286.55 m (EL. 338.45 m) are formed by limestone; (b) The other sections down to the bottom, 341.6 m (EL. 283.4 m) are formed by matrix of Ophiolitic Melange; (c) The groundwater level was approximately 120 m in depth (EL. 505 m), which was measured during drilling works.

The results of borehole SK-108b are as follows: (a) The section from 5.0 m to 196.75 m (EL. 287.44 m) is limestone; (b) The section from 196.75 m to 201.2 m (EL. 282.99 m) is matrix layers of the Ermenek Ophiolitic Melange.

Matrix layers are D, CL to CM class rocks. Such rocks are generally unsuitable for the construction of an underground power house. Rocks having appropriate conditions are only limestone in this area. The location of the power house was conceived in the elevation range between about 309 m and 349 m.

The borehole SK-108b shows that the depth of limestone bottom is 196.75 m or EL. 287.44 m. In the borehole SK-106 located on the Erik river, limestone thickness is 186.70 m, and the bottom of limestone is at EL. 182.85. These bottom elevations at SK-108B and SK-106 are lower than that of the bottom elevation of the power house (EL. 309 m). Therefore, the power house can be placed in the limestone block between the SK-108b and SK-106.

Rock properties of the limestone may be similar to those in the I-C dam site (Refer to Sub-section 4.3.2).

(4) <u>Headrace tunnel</u>

(A) <u>Geology along tunnel route</u>

The total tunnel length is about 9,042 m and the tunnel diameter is 6.1 m. The geological conditions along the headrace tunnel are schematically shown in Plate G14.

The tunnel will penetrate through limestone for about 6,300 m; matrix layers of the Ermenek Ophiolitic Melange for about 2,700 m. These limestones are all blocks in the Ermenek Ophiolitic Melange, and the matrix layers of the Melange are composed of schist, sandstone, siltstone, conglomerate, ophiolite, serpentine and so forth.

The limestone blocks are all hard and massive rocks, which correspond to CH to B class of the rock classification (K.Kikuchi Et.al) in slightly weathered to fresh rocks. The matrix layers are medium hard to soft rocks which correspond to CM to CH class in fresh rocks. In the boring core sample, the rocks are in general highly cracky and partly fractured (refer to Table 5.5).

A mineralogical study was performed for green schist and serpentinite by X-ray diffraction method for the purpose to examine swelling tendency of these rocks. Samples were taken from outcrops near the power house and from the borehole SK-102. The results suggest that these rocks have nil or very rare amount of minerals which have a swelling tendency.

(B) <u>Groundwater level</u>

The groundwater level in a limestone block of the Nadire Formation situated at the entrance of the headrace tunnel is almost the same with the river water level (about EL. 500 m). Other measurement records are only in the boreholes SK-101 and SK-102, which are 20.0 m in depth (EL. 960.61 m) in SK-101, and about 115 m (EL. 500 m) in SK-102.

Many springs were observed along the tunnel route. The source of these springs is estimated to be limestone of the Ermenek Formation, which is distributed widely on the higher mountain slopes in the southern side of the tunnel route. These springs suggest that the groundwater level is high in general, and rises in parallel with the ground surface towards the plateau on southern basin boundary. However, the water levels in limestone blocks are supposed to be rather low in comparison with those in matrix layers because of the karstic character. In those blocks which have no direct contact with the limestone of the Ermenek Formation, the water levels would be much lower even if these blocks are located at a high elevation in the matrix layers.

(5) Surge tank and pressure shaft

The headrace surge tank site is located in a limestone block of the Nadire Formation, which corresponds to CH to B class of the rock classification (K.Kikuchi Et.al.) in slightly weathered to fresh conditions. The foundation will be slightly weathered to fresh rocks (4.5 km/s in seismic velocity). The upper section will be in the rocks which have seismic velocity of 3.1 to 3.3 km/s or 2.0 to 2.3 km/s, and correspond to CH to CM class.

The pressure shaft route will pass limestone of the Nadire Formation in the uppermost and lowermost sections; matrix layers of the Ermenek Ophiolitic Melange in the middle section. The rocks are estimated to be in slightly weathered to fresh conditions for the whole section.

(6) Tailrace tunnel

The geological conditions along the tunnel route are shown in Plates G17 and G18. The tunnel will pass limestone for 1,050 m from the power house; matrix layers for next 150 m; marl in the remaining part up to the outlet. According to the seismic exploration of line PC, the limestone in all the tailrace tunnel section is slightly weathered to fresh (4.5 km/s in seismic velocity); matrix layers and marl (3.6-3.7 km/s) are fresh except for the outlet portion. For the marl section, a medium to heavy supporting system will be required because of its slaking tendency. A heavy supporting system will be required for several 10 m in length from the outlet portal.

(7) Landslide area in the dam site I-B

The geological conditions are shown in geological profiles for the seismic exploration lines LA, LB and LC in Plates G19, G20 and G21 respectively. The main conditions which cause landslides in relation to the dam impounding are generally as follows (M.Watari, Chairman of Japan Landslide Society):

(A) Topography and geology show features of historical landslide activities.

(B) Steep slope inclination. In many cases of landslides observed in the past, the slope inclination was more than 20 degrees in general.

(C) Rapid drawdown speed of reservoir. Most of the past landslides occurred during a rapid drawdown of reservoir water level at a rate of more than 2.0 m/day.

Although the topography and geology in this site show historical landslide features, the slope inclination is about 10 degrees on an average, and the drawdown speed of the Ermenek reservoir is expected to be in a order of 0.1 m/day, being much slower than the above rate of 2.0 m/day. It is judged that no large scale landslides are likely to occur during and after impounding of the reservoir.

(8) Erik intake, diversion tunnel and power house

(A) Erik intake weir

The Erik intake weir site is located at an immediate upstream point of a landslide which exists in the Erik river basin. The geological condition is shown in Plate G22. The intake site is formed by hard and massive limestone of the Kükürce Formation in the Ermenek Ophiolitic Melange. The moderately weathered limestone corresponds to CH to CM class in the rock classification, and has sufficient strength as the weir foundation. Excavation of 1 to 3 m in depth will be required to obtain CM to CH class rocks. The riverbed is covered by sand and gravel layers of several meters in thickness. These deposits will have to be removed for the weir construction.

(B) Erik diversion tunnel

The tunnel route is proposed in rather deep portion of the mountains to detour the active landslide area. The geological conditions are schematically shown in Plate G23. The rocks are limestone and matrix layers of the Melange. Tunnelling works will need light to medium support works for the matrix layer sections, and no support or light for the limestone sections.

(C) Erik power house

The power house site is located in the matrix layers of the Melange. The surface is covered by thin talus (1 to 3 m) deposits. Foundation rocks are alternation of sandy limestone and siltstone, rarely schist, sandstone and conglomerate. Strength of these rocks will be sufficient for the power house. In the upper slope of the power house approximately above EL. 740 m, hard limestone of the Azitepe Formation outcrops well. The proposed headtank and outlet of the diversion tunnel will be located in this limestone.

(9) Reservoir area

(A) General condition

The geology of reservoir area is mostly composed of sedimentary rocks, such as marl, sandstone and conglomerate of the Tertiary Görmel Formation, and partly matrix layers and limestone blocks of the Ermenek Ophiolitic Melange. These rocks except limestone blocks are generally low pervious without solution cavities. There are no leakage problems in the area which is formed by the Görmel Formation and matrix layers of the Melange.

Limestones are seen only in the dam site area and in the backwater area. Limestone in the dam site will be treated by the curtain grouting of the dam.

No serious slope stability problems are likely to occur in all the reservoir area, because slopes in the surrounding area of the reservoir show very gentle inclination in general.

(B) Leakage from backwater area of reservoir

Alternation layers (siltstone, sandstone, claystone and limestone) of about 100 m in thickness are distributed on the right bank of the Ermenek river with bedding of approximately EW/20-40deg.N. These layers are low pervious in general. On the right bank, this layer will act as a barrier against groundwater flow towards south.

The northern area of the river is formed by very wide mountainous plateau having an elevation higher than 1,000 m, where the limestone block is surrounded by the matrix layers, which are covered by Miocene karstic limestone. Also, many springs are widely distributed in this area. This area is considered to have very high potential of groundwater.

In the upstream reaches of this limestone block, the Nadire and Kapiz springs exist and suggest that the area has high potential of groundwater. Also thrust fault which may be located just upstream of these springs with south-north trend would act as a barrier against groundwater flow towards west.

Consequently, the groundwater levels in both the banks are estimated to be higher than the proposed reservoir water level in general. Therefore, possibility of leakage through this limestone will be very low.

5.3.4 Seismicity

Four earthquake zones are seen in the Anatolian Peninsula; northern, central and southeastern Anatolian zones, and Aegean-Marmara zone to the west as shown in Fig. 5.4. The Project site is located to the south of Central Anatolian zone, and is one of the least seismically active areas in the Anatolian Peninsula.

Earthquake data of 695 events having magnitudes of more than 4.5 observed in the period from 1901 to 1987 were obtained from the Istanbul Bosphorus University. The epicenters are located within a radius of about 500 km. Epicenters of the collected data are shown in Fig. 5.5. The study was performed by 3 methods: (a) Study of probable earthquakes in the surrounding area of the site; (b) Kawasumi's method; (c) Study of maximum credible earthquake.

(1) <u>Probable earthquake in the surrounding area of the site</u>

Seismic frequency and risk were estimated for 2 magnitudes of 4.5 and 7.2 by the least square method. The magnitude of 7.2 corresponds to a return period of 193 years. A probability that an earthquake of magnitude 7.2 occurs in a 100 years is estimated at 41 per cent (see ANNEX-A for details).

The frequency and risk were calculated also by a probability method: the magnitude of 7.2 corresponds to a return period of 56 years. A probability that an earthquake of magnitude 7.2 occurs in a 100 years is estimated at 83 per cent (see ANNEX-A for details).

(2) <u>Probable Maximum Acceleration by Kawasumi's method</u>

The probable maximum acceleration at the Project site was calculated by the Kawasumi's method. The maximum acceleration at site in a return period of 100 years was estimated as follows:

A = 12 gal = 0.01 g

(3) Maximum Credible Earthquake

The earthquakes of 7 events were selected to estimate the maximum credible earthquake and to determine the seismic coefficient for design. The peak ground accelerations of the 7 earthquakes were estimated using (a) the Estiva's method; (b) Cornell's method and Kawasumi's method, which had been used in the EIE's analysis. The results are shown in Table 5.6. The accelerations estimated except for the Project Earthquake are less than 0.0141g, and the acceleration estimated for the Project Earthquake shows the highest ground acceleration from 0.0361 to 0.2932g.

(4) Seismic coefficient for design

The probable maximum acceleration by Kawasumi's method is 0.01g in a return period of 100 years, and the Project Earthquake shows the highest ground acceleration from 0.0361 to 0.2932g in the maximum credible earthquake study.

Taking these conditions into account, the seismic coefficient of 0.05 to 0.1g is judged reasonable for design of the Project structures.

(5) Reservoir-induced earthquakes

Reservoir-induced earthquakes were observed generally in those reservoirs which had maximum water depths of more than 100 m. Magnitudes of these earthquakes were generally of small to medium scale, being less than 6.

Since the maximum depth of the proposed Ermenek reservoir will be about 180 m, there will be a general possibility of the reservoir-induced earthquakes. However, the ground acceleration, which will shake the proposed dam site in case of these earthquakes, would be lower than that of the Maximum Credible Earthquake. No specific consideration will be required for the determination of seismic coefficient for the design.

5.4 Construction Materials

5.4.1 Impervious core materials

As illustrated in Plate G27, potential borrow areas of the core materials investigated are Aa, Ab, Fa, Fb, C, D, B and I along the Ermenek main stream, and Ea, Eb and Ec along the Zeyve river. Results of the laboratory tests are summarized in Table 5.3. The quantity of materials in these borrow areas was estimated to be 7.7 million m^3 in total (see ANNEX-A for details).

The earth materials obtainable in the borrow areas Fa, Fb, C, D, I, Ea and Eb are suitable for both the ordinary core and the rock contact core. The earth materials obtained from the borrow area Ec are usable for the ordinary core, but not suitable for the rock contact core. The earth materials obtainable in the borrow areas Aa and Ab are not suitable for the core.

It is recommended that the priority be given to the borrow areas Ea and Fa to obtain the core materials for both the ordinary core and contact core taking account of the shorter hauling distance than the others. The borrow areas Eb and C should be considered as alternatives in the wet season.

Recommended design values of the core material are as follows.

Wet density	:	1.90 t/m^3
Saturated density	:	1.95 t/m^3
Cohesion		
Cuu	:	6.0 t/m^2
C'	:	1.5 t/m^2
Friction angle		
θuu	:	10 deg.
θ'	:	28 deg.
Permeability	:	1×10^{-5} cm/s

5.4.2 Sand and gravel materials

Potential borrow areas of the filter materials and concrete aggregates investigated are Ga, Gb, Gc, Gd and Ge. These are riverbed deposits and alluvial terrace deposits along the Ermenek river, as shown in Plate G27. The test results are summarized in Table 5.3. The quantity available in the borrow areas is estimated to be about 1.1 million m^3 in total (see ANNEX-A for details).

The materials from all the borrow areas are suitable in quality for the filter materials and concrete aggregates, although appropriate gradation control and washing will be required.

5.4.3 Rock materials

The location of quarry site is shown in Plate G27. The quantity available is estimated at 2 million m^3 for concrete aggregates and 13.5 million m^3 for embankment of a rock fill dam. Table 5.3 shows a summary of the laboratory tests of quarry rocks.

The test results indicate that the rock materials obtainable in the quarry site are generally acceptable in quality. The rock materials obtained from the quarry site can be used for many objectives such as the rockfill material, transition material and filter material for the dam embankment, and the concrete aggregates.

5.5 Hydrology

5.5.1 Climatic features

General characteristics of the climate in and around the Göksu river basin are as summarized below:

(1) A remarkable difference in climate can be seen between the coastal area along the Mediterranean Sea and the highland area in the central inland. The seasonal pattern of precipitation in the coastal area is generally influenced by the Mediterranean climate. Precipitation, either rainfall or snowfall, mainly occurs in winter and spring when the depression is activated. The climate in summer is dry. A continental climate prevails on highland in the central part of Turkey, and causes a cold wind and a snowstorm in winter and hot and dry climate in summer.

(2) The main cause of rainfall and snow in and around the Göksu river basin is mainly the western depression, which is moving to the Middle East or Turkey from the west, Mediterranean Sea or North Africa. There are two types of depression. One runs fast through the southern part of mountainous area in Turkey, the other runs slowly and is sometimes stagnant near the Cyprus Island. The former depression occurs mostly in winter, the latter occurs mostly in spring.

(3) The Mediterranean climate is dominant at the lower elevations of the Ermenek river basin. The elevation rises rapidly towards the upstream basin and the climate is likely to be influenced by the altitude. The mean ground elevation (1,600 m) of the basin indicates that the basin climate falls into a transitional zone from the Mediterranean climate to the continental climate.

(4) Heavy rainfall occurs along the coastal areas, where it records from 1,000 to 1,600 mm a year. The Ermenek river basin is sheltered from the south- and northwest winds by the coastal mountain range. The average annual rainfalls at the meteorological stations in the basin are observed between 500 and 900 mm. Precipitation in the basin increases towards the west along the river. This areal variation is in agreement with the prevailing wind directions from the south and northwest during storm and with the general topographical features of the basin. An isohyet of the mean annual rainfall is shown in Plate P2.

(5) Maximum wind velocity recorded during the past 20 years is 23 m/s at Mut. The direction of the dominant wind is northwest. Temperatures in the basin fluctuate widely in a year. The annual mean temperature in Ermenek is 11.8°C and the extreme maximum and minimum temperatures are 39°C in July and -15°C in January. The climate features are shown in Figs. 5.6 and 5.7.

5.5.2 Rainfall

(1) <u>Basin precipitation</u>

Basin precipitation of the Ermenek river at the Ermenek dam site was 946 mm/yr on an average for the past 23 years from 1965 to 1987. The monthly precipitation pattern is shown in Fig. 5.8. Approximately 80 per cent of the annual precipitation occurs during the winter and spring season; December to May.

(2) Rain storm

In the Göksu river basin a severe storm was observed on January 30 to 31, 1975. The rainfall duration was about 48 hours. The maximum rainfall depth of this storm over a catchment area of 2,156 km^2 (at the Ermenek dam site) was 102 mm for the duration of 24 hours.

5.5.3 Runoff

(1) Stream gauging stations

DSİ has been operating a stream gauging station No. 17-14 since 1965 but with a operation gap for 3 years from 1969 to 1971. The gauge height has been read once a day at 8:00 a.m. on a staff gauge installed beside the Görmel bridge. However, the flow measurements have been made near Çavusköyü village, 1.75 km upstream from the bridge.

EİE installed in 1985 a new gauging station, No. 1723 (Çavusköyü), at the place of flow measurements of Station 17-14 above. The gauge height reading has been continued twice a day at 8:00 and 16:00 on the staff gauge installed at this place (see Fig. 5.12 for the location).

(2) Snow-melt effect to water level at Stations 17-14 and 1723

Through the hydrological review of the water level, river flow, discharge rating curve and rainfall records of the Göksu river basin, the following issues were found:

(A) Mean runoff coefficients estimated for the Ermenek basin based on the 23 years' records from 1965 to 1987 differ much between the basins upstream and downstream of Station 17-14 as listed below:

- at Station 17-14 (dam site)	:	0.81
- Dam to confluence with Göksu	:	0.39
- Ermenek at confluence	:	0.67
- Göksu at confluence	:	0.59
- Overall Göksu basin	:	0.57

The high value of 0.81 at Station 17-14 and the low value of 0.39 for the downstream basin imply an overestimate of the runoff at Station 17-14.

(B) The mean runoff of Station 17-14 was about 19 per cent higher than that of Station 1723 for the latest 3 years from 1985 to 1987.

(C) However, it is judged that both the discharge rating curves of Stations 17-14 and 1723 are well consistent to each other, and are highly reliable.

(D) The water levels at Station 1723 fluctuated much everyday in the spring season from March to May. The morning water levels were mostly higher than the afternoon levels. This daily fluctuation continued sometimes for one month without exceptional days, but disappeared after June. This implies that the fluctuation in the spring months has the period of 24 hours.

To confirm this phenomenon, the river water level was observed more frequently in April 1990 on the staff gauge, which had been installed by EİE between the

Görmel bridge and the I-Ca dam site. As shown in Fig. 5.13, the water level showed a clear fluctuation having a period of 24 hour.

It is judged that in the snow-melting months of spring the morning water level at the dam site is higher than the afternoon level, being affected by the snow-melt discharge. The daily water level and discharge records of Station 17-14 were over-assessed in the spring season because of the gauge height reading of only once a day at 8:00 a.m.

The runoff records of Station 17-14 were then reduced to 0.84 times, with reference to that of Station 1723. The runoff coefficient at Station 17-14 became 0.73 after this adjustment, and is considered to be reasonable when compared with a new figure of 0.59 for the downstream basin and figures for the other sub-basins.

The monthly runoff pattern is shown with the rainfall pattern in Fig. 5.8. It clearly indicates the runoffs in March, April and May are mostly fed by snow-melt.

(3) Long-term average runoff

The 20 years' runoff series adjusted above (1965-1968, 1972-1987) was supplemented for the 3 missing years. An average inflow of the Ermenek reservoir was estimated at 47.5 m^3/s for the 23 years from 1965 to 1987.

Meanwhile, Station 1720 located on the Göksu branch stream has the longest period of flow records among the stations in the Göksu basin. The 42 years' runoff records from 1946 to 1987 of this Station show that the above 23 years' period was a rich water period compared to the preceding 19 years' period from 1945 to 1964 as summarized below:

Period	Years	Mean Runoff (m^3/s)	Mean Runoff (%)
1946-1964	19	41.2	88
1965-1987	23	51.1	110
1946-1987	42	46.6	100

In order to get rid of a potential overestimate of the mean inflow on the long-term basis, the 23 years' inflow series derived for the dam site was extended for the dry period of 19 years based on the runoff records of Stations 1719 and 1703 (Kirkyalan).

The long-term mean inflow was thus estimated at 43.0 m^3/s for the period from 1946 to 1987, which are about 91 per cent of the estimate of 47.5 m^3/s for the recent 23 years. The historical changes of estimated annual inflows are shown in Fig. 5.14.

(4) <u>Runoff of the Erik river</u>

The mean annual runoff of the Erik river was estimated using the relationship of monthly runoffs at Stations 17-14 and 1715 (Erik-Ilisu), commonly available for the 3 years of 1966, 1970 and 1971. The monthly runoff pattern in 1970 was adopted as a typical pattern in view of its lowest dry season flow among the three years.

5.5.4 Flood

The annual maximum floods recorded at Station 17-14 for the 18 years are listed in Table 5.17. Frequency analyses were made on these flood peak flow and volume of 6 durations; 1 day, 2-day, 3-day, 5-day, 7-day and 10-day. The analysis was made in accordance with a method of Log Pearson Type III. The results are presented in Tables 5.18 and

5.19, and are summarized below for the flood peak flow:

Return Period (yr)	Peak Flow (m^3/s)
2	600
5	900
10	1,100
20	1,400
30	1,500
50	1,700
100	2,000
200	2,200

The probable floods of Erik river were estimated using catchment area - specific discharge relationship given in Fig. 5.15. The 100-yr probable flood at the Erik intake weir site is estimated at 400 m^3/s.

5.5.5 Probable maximum flood

In the Göksu river basin where a large flood may result from a combination of snow-melt and rainfall floods, it is necessary to examine the flood peak flow and volume not only for a rainstorm flood but also for a combined flood of rainfall and snow-melt. Rainstorm would cause a Probable Maximum Flood (PMF) in the winter season when there is much rainfall but less base flow. The base flow by snow-melt discharge would cause PMF in the spring season when accompanied by a rainstorm. Thus PMF was derived for 2 critical seasons; winter and spring.

PMF is derived by converting a probable maximum precipitation (PMP) into a flood hydrograph using a unitgraph and by adding base flow adequate to the season considered.

(1) PMP for winter

The PMP for the Ermenek basin at the proposed dam site ($Ca=2,156\ km^2$) was derived by maximizing Depth-Area-Duration Curves (DAD) of the recorded maximum storm in January 1975. A moisture maximization factor was obtained to be 1.8 (see ANNEX-C for details).

The PMP over the Ermenek basin thus obtained is summarized below:

Duration (hr)	Depth of 1975 Storm (mm)	PMP (x1.8) (mm)
1	22	40
6	68	122
12	90	162
24	102	184
48	162	292
96	253	455

(2) PMP for spring

PMP for spring was obtained for the typical snow-melt month of April by reducing the PMP for January based on the seasonal maximum daily precipitation curve (see ANNEX-C for details). The PMP for April is approximately 75 per cent of that for January.

(3) Unitgraph

Unitgraph for the Ermenek basin was derived in accordance with the method of dimensionless graph. It is obtained for the condition with the Ermenek reservoir. A reservoir will in general increase the flood peak flow compared to the natural condition because it will shorten the basin lag.

Two dimensionless graphs were derived based on the flood hydrographs recorded at Station 1714 (Kayraktepe dam site) in January 1971 and February 1975.

Another dimensionless graph was also derived based on a flood hydrograph observed at the Seyhan dam on the Seyhan river between March 27, 1980 and April 7, 1980. This flood was an unusually large flood ever observed in the basins facing the Mediterranean Sea.

The unitgraph was derived for unit rain of 1 mm and unit duration of 1 hour. This unit duration was selected with reference to the estimated range of basin lag between 2.5 and 10.5 hours. These basin lags were estimated for 3 sub-basins and for 3 floods as summarized below:

		Basin Lag t'p (hr)		
Sub-basin	$L\bar{L}/S^{0.5}$	St. 1714 Feb. 1975	St. 1714 Jan. 1971	Seyhan D. Mar. 1980
A	129	10.5	8.5	5.5
B	53	6.5	5.5	3.5
C	14	5.5	4.5	2.5

These lags are compared with empirical curves in Fig. 5.16. The figure suggests that even the estimates given by the Seyhan flood are not very short. However, the estimates given by the January 1971 flood at Station 1714 were adopted in this study to derive PMF, giving weight to the flood recorded in the basin and adopting the shorter lags between the two floods at Station 1714 to be conservative in terms of flood peak flow.

The unitgraph was constructed by combining 3 unitgraphs for 3 sub-basins as shown in Fig. 5.17.

(4) Rainfall loss

Initial rainfall loss is neglected assuming that the whole Ermenek basin is saturated by antecedent rainfalls in such an extreme storm like PMP. While the retention loss rate after the saturation is simply assumed to be constant at 2.0 mm/hr since there is no data available.

(5) Base flow in winter

Based on the daily discharge records of Station 17-14, the maximum base flow probable in the winter season is assumed to be 100 m^3/s.

(6) Snow-melt flow in spring

Snow-melt runoff and its maximum rate were studied by EİE using the degree-day method. The results of this study were reviewed and adopted in this PMF study.

(7) Probable maximum flood

The PMP for 96-hour duration is rearranged to hourly rainfalls with its peak at one fourth from the end of the duration maintaining the depth-duration relation.

Both PMP and resultant PMF for January and April are shown in Figs. 5.18 and 5.19, and their peak flows are summarized below:

(Unit: m^3/s)

Month	By PMP	Snow-melt plus base flow	Total
January	5,800	100	5,900
April	4,100	1,300	5,400

5.5.6 Sediment

EİE has carried out suspended sediment sampling at Station 1723 (Çavusköyü) since 1985. Sediment rating curve is generally approximated by a straight line on a full-log-paper as shown in Fig. 5.20. The curve can be expressed by the following equation:

$$Q_s = 0.405\ Q^{1.65}$$

Suspended sediment transport of the Ermenek river is estimated based on the daily runoff records at Station 1723 from 1985 to 1988 using the sediment rating curve above with some adjustments for the period of runoff series used. Mean sediment inflow of the Ermenek reservoir including bed load is estimated at 830 m^3/day, or 130 m^3/km^2/yr. This corresponds to an annual denuded depth of the land of 0.13 mm.

Assuming that a trap efficiency of sediment inflow by the Ermenek reservoir is 100 per cent in view of its large reservoir capacity, the mean annual reservoir sedimentation is estimated at 0.3 MCM. After the 100 years operation, the total sediment deposit volume would be in the order of 30 MCM, which are negligibly small compared to the dead reservoir capacity of 1,190 MCM.

5.5.7 Water quality

Water sampling of river flow and its chemical analyses have been carried out by EİE at Station 1723 since 1985. The water of the Ermenek river was slightly alkaline, with the average pH value of 8.0.

A study made on the relation between pH and corrosion of steel equipment and facilities of hydroelectric power plants in Japan indicated that the acid water with the pH value below 4.5 mostly had caused corrosion.

It is judged that the hydraulic turbines of the Project will be free from corrosion problem.

5.6 Access to the Project Site

5.6.1 Existing ports

There are 2 international seaports conceivable for the Ermenek Project; Mersin and Antalya, both facing the Mediterranean Sea. In view of the required road distance from the port to the Ermenek site, the Mersin port is more advantageous.

The Mersin port was used also for unloading of the heavy equipment for the Keban, Karakaya and Atatürk hydroelectric power projects, and is planned for the Kayraktepe project. Unloading facilities of the Mersin port is sufficient to handle the heavy equipment to be imported for the Project.

5.6.2 Existing road network

The existing road network is shown in Plates P1 and P29. All the following roads were designed for the total load of 36 ton including vehicle weight:

Mersin - Silifke	(84 km)
Silifke - Mut - Ermenek	(171 km)
Silifke - Gülnar - Ermenek	(159 km)
Silifke - Anamur - Ermenek	(226 km)

5.6.3 Conceivable transportation route

For transportation of the materials and equipment to the Project site, the following 2 routes are conceivable depending on the origin of goods and unloading sites:

(1) <u>Mersin port to Ermenek power station site</u>

Such imported equipment as generating equipment and transformers will be unloaded at Mersin Port. These will be transported through the Mersin-Silifke-Mut-Ermenek road and an access road. This access road will be constructed branching from the Mut-Ermenek road towards the power station site across the Ermenek river. The road length is about 234 km in total.

(2) <u>Mersin port to Ermenek dam site</u>

Those imported materials and equipment for the use around the dam site may be transported to the Ermenek dam site through the Mersin-Silifke-Gülnar-Ermenek road for its shorter length. The road length is about 220 km.

5.6.4 Required improvement works of roads

The conditions of Silifke-Mut-Ermenek road and Silifke-Gülnar-Ermenek road are presented in Tables 5.20 and 5.21.

The Mut-Ermenek road will need the following improvement works for transportation of generating equipment and transformers to the Project site:

(1) Detours to cross the river at a bridge located at 6 km from Mut and at the Kadi Bridge located at 8 km from Mut.

(2) Improvement of alignment and widening of road width for about 2 km in total in the sections between 35 and 45 km from Mut.

The Gülnar-Ermenek road will need improvement works of curves and slopes of the road for a section of several km located to the northwest of the Olukpinar bridge.

CHAPTER 6. ENVIRONMENTAL ASPECTS AND COMPENSATION COST

6.1 Existing Environmental Conditions

6.1.1 Population

The population density in the Konya, Karaman, Içel and Antalya provinces are low but growing fast compared with the whole of Turkey indicating rapid urbanization in recent years. The population in the Göksu river basin in 1985 was 250,000 or 24/km^2 and that in the Ermenek river basin was 70,000 or 29/km^2, the sparsely populated showing rural characteristics. There are only a few urban centers such as Ermenek, Hadim, Bozkir, Karaman, Mut, Gülnar and Silifke.

6.1.2 Economy

Per capita gross regional domestic product (GRDP) in 1986 was TL552,000 in the Konya province (divided into the Konya and Karaman provinces in 1989), TL951,000 in the Içel province and TL670,000 in the Antalya province respectively, while it was TL757,000 in the whole Turkey. Manufacturing industry in Mersin has sustained a high GRDP in the Içel province while the four Provinces as a whole was agricultural.

Wheat is the most dominant crop in these provinces. Important industrial crops are cotton on the Mediterranean area but grapes, apple, pear, peach and cherry are grown in the interior.

Livestock is extensively grazed or individually fed on a small scale. In the Göksu river basin, there are small lumber industries based on pine trees or some tree plantations.

There are lignite coal mines near the proposed reservoir area. Akpinar open pit mine operating at ELs. 760-820 m will be closed in a few months. Komur Isletmeleri A.S. is located 2 km northwest of the confluence of the Ermenek river and the Küçüksu river. It has a shaft operated at a loading ramp at EL. 600 m. This will be closed within next ten years. The Coal Mine Corporation produces 55,000 tons/yr from a reserve of 800,000 tonnes, employing 218 workers and 6 other staff. Presently the mining shaft reaches, from the surface of EL. 700 m to a bottom of EL. 300 m.

There are concrete factory, sugar company and a paper mill in Silifke. An aluminium smelting plant is operating in Seidişehir and there is a cement factory in Konya.

The Göksu river basin is not within the tourism networks in Turkey though tourists visit it.

6.1.3 Land use

More than half of the area of Içel and Antalya provinces is covered by forest but most areas are utilized for agriculture in Konya and Karaman. The land use in Ermenek district in 1988 is shown in Table 6.1. Forest covers 69.4 per cent of the land; cultivated land is 13.8 per cent; and pasture/meadow occupies 13.5 per cent.

6.1.4 Public health

There is a hospital in the Ermenek city with 50 beds occupied 71 per cent in 1988. There are six health service units conducting simple treatment of outpatients and preventive injections. There are major hospitals in Konya and Mersin. Table 6.2 shows a record of patients hospitalized between January and October 1989 in the Ermenek hospital.

6.1.5 Sanitation

The Göksu river basin is rich in springs and groundwater of good quality and most drinking water supplies depend on them. In Ermenek city, 2,796 households are served by public pipes fed by spring water. Daily water consumption is estimated to be 50 litres per capita. There is no sewerage system in the Göksu river basin.

6.1.6 Topography and geology

The Ermenek river basin is located in the middle of the Taurus range with peaks of ELs. 1,500 to 2,000 m. The valley of Ermenek river is 700 m deep below Ermenek city. Adjacent hills rise 300 m above the city. The Lower Miocence Görmel Formation forms a wide valley in the proposed reservoir area, while the Upper Miocence Ermenek Formation characterizes the limestone plateau and cliffs with solution cavities and springs. An old limestone block in the Cretaceous Ermenek Ophiolitic Melange forms the deep Görmel Gorge.

Upper catchment of the Ermenek river is fairly well covered by vegetation except where calcareous rocks crop out. In the lower part of the upper catchment there are small poplar plantations and fruit orchards of olive, peach, apple, fig and grape.

6.1.7 Surface water

Most tributaries of the Ermenek river are predominantly fed by spring water. It is estimated that 60 to 70 per cent of the precipitation drains into the river.

The sediment concentration in the river flow is very low because of the rocky terrain and vegetation cover. The water quality data in Table 6.3 show that the river water is almost devoid of contamination. Relatively high concentration of Ca/Mg, Co_3 and HCO_3 is due to the calcareous rocks prevailing.

6.1.8 Terrestrial fauna and flora

There are about 120 species of wild mammals in Turkey, but much less species live in the Ermenek river basin. Dominant in number are moles, rabbits, rats, voles and squirrels. Table 6.4 shows bird species whose presence has been confirmed by the Ermenek Regional Office of the General Directorate of Forestry.

Poplar is the predominant hardwood in the Ermenek river basin and conifers such as black pine and various junipers are often observed. Table 6.5 lists tree species confirmed in the river basin by the above-mentioned Office.

6.1.9 Aquatic fauna and flora

Fish species which may be found in the Göksu river system are listed in Table 6.6.

Aquatic flora are very limited in the Project area, because the Ermenek river seldom has still waters and marshy areas. It is assumed that the population of phyto-planktons is small.

6.2 Results of Compensation Survey

6.2.1 Basis of survey

The compensation survey for the proposed Ermenek reservoir area was conducted by EİE assisted by the agricultural engineers of the Ermenek District, Ministry of Agriculture, Forestry and Rural Affairs, based on the Manual for Expropriation Studies, which was prepared by DSİ in 1984, in conformity with the legal requirements of Law Number 6200 and Law Number 6830.

The survey area covered 7,367 ha between ELs. 510 m and 750 m.

6.2.2 Population and land use

The proposed Ermenek dam, having a crest elevation of 680 m, will have a reservoir surface area of 4,843 ha at HWL of 675 m. The portions used for agriculture, including poplar planting, are located at lower altitudes and cover 844 ha of land. Forests are found at altitudes higher than 600 m covering 2,086 ha, or 43 per cent of the total reservoir area. The remaining 1,913 ha (40 %) are water surface, barren land and settlement. The total number of people that will be affected was approximately 500, of which about 200 lived in the village of Çavusköyü. The remainder are living in scattered settlements.

6.2.3 Land value and compensation cost

The average land value in the reservoir area in 1988 was seen to be in the range of TL 8 million to 15 million/ha or US$4,760 to 8,920/ha (US$1.00 = TL1,681.61 in 1988). A typical house is constructed of dry stone masonry, wood framing and compacted dirt roofs. There is a mosque, a primary school and a building of the General Directorate of Forestry in Çavusköyü.

The main economic activity of the inhabitants is mixed farming with some animal husbandry. Some people find employment in Ermenek city and lignite coal mines. Income generation is mostly based on agricultural activity. On a per hectare basis mixed orchards and vegetable farming provide the highest income. Total net income is estimated to be TL1,450 million or US$0.86 million, on the basis of the rate of return on land used for various purposes.

The expropriation value excluding movable assets is assessed to be TL22,800 million or US$9.8 million, based on the above-mentioned considerations.

6.3 Possible Environmental Impacts

6.3.1 Socio-economic impacts

For the implementation of the Project, resettlement of about 500 people who are living in the proposed Ermenek reservoir area will be needed. In recent cases of expropriation by the Government, most of those displaced chose to move out of the region and eventually became urbanites. It has been common, especially since the early 1950s, for outmigrants to initially settle in urban centers close to their villages and then to move to the larger metropolitan centers such as Istanbul, Ankara, Izmir and Adana. Of the 500 population that would be resettled upon the completion of the proposed dam, about 200 persons live in the village of Çavusköyü. Based on on-site investigations and interviews with local officials, the economically active population, of 12 years and older, number about 194 and most of these persons work on their own farmland and some are employed at the local lignite coal mines and at the district center of Ermenek. It is believed that the regional labor market is capable of absorbing the displaced working population. The absorption into the region's urban centers should not pose much difficulty as the region has several major thriving urban centers such as Konya, Antalya, Mersin and Tarsus. Some of the displaced population may move to large urban centers like Bursa, Adana, Izmir, Ankara and Istanbul, following the resettling pattern observed in the past.

The Project will provide employment opportunity primarily for common laborers from the preconstruction stage through the operation stage but mostly in the construction stage. Some people removed from the reservoir area will find jobs in the Project works. Service and other commercial activities will be benefited by the influx of people in the preconstruction stage and more significantly in the construction stage. This kind of influence will be induced especially in urban areas such as Ermenek, Hadim, Mut, Silifke, Mersin, Karaman and Konya.

The acquisition of farmland and removal of people from the proposed reservoir area will reduce agricultural activities, but this adverse effect will be small in relation to the district's agriculture, and it will be partly compensated by commercial or other activities of the people moved from the reservoir area to Ermenek city or other urban areas.

In accordance with the objective of the Project, the electricity supplied by the Project will contribute significantly to the industrial and regional development of Konya and Karaman provinces.

The created water surface will provide opportunities for development of aquaculture. Convenient and low cost transportation on the lake will encourage trading activities. Tourism may also be promoted.

The resettlement of farmers will inevitably involve some land tenure problems.

No change is expected in health conditions, except possibly for some water-born disease on the lake shore though only to a minor extent.

6.3.2 Physical impacts

The creation of a still water body will have various influences on the environment with the progress of the Project.

The influence of the reservoir on the climate will be negligible if any. The regulation of river flow by the reservoir will generally be beneficial to external activities because low flow will be augmented and flood discharge will be cut down.

Groundwater levels may change in the vicinity of reservoir. The existing shaft of the Coal Mine Corporation has no water problem even though its bottom is more than 300 m below the present groundwater level. This shaft might be affected by an elevated groundwater level in the future, though the resulted loss will be marginal compared with the production in TKI mine.

The organic content will tend to increase due to decomposition of organic material flowing in or being submerged. This may cause an increase in plankton and other aquatic life. Judging from the intact water quality and sparse vegetation, these changes will progress quite slowly. No change of this kind has been observed in the Oymapinar reservoir which has been operated since 1984. Sedimentation once trapped in a reservoir may cause prolonged turbidity in the downstream river course sometimes to the extent of affecting fish culture. This will not be the case in this Project, where sedimentation is quite small.

There are indications of old landslides on slopes of proposed reservoir area, but it is judged that impounding of reservoir will not cause any landslide because the existing slope of ground surface is already gentle and the expected movement of reservoir water surface will be very slow.

Vegetation in the Project area will be affected to some extent due to construction of access roads, quarry sites, quarters and spoil banks during the construction period and due to the submergence in the reservoir during impounding.

The landscape of canyon in the Project area will be altered to that of reservoir water surface. This may be felt to be more beautiful by some people and the reverse by others.

The eutrophication of the reservoir may be a crucial problem if it is induced by largely increased human activities in the future.

6.3.3 Ecological impacts

Little impact is foreseen on terrestrial fauna and flora, except that the territory of small mammals and breeding of some birds may be affected by creation of a large water surface.

There will be the influence of construction of dam on the fish species, but this must already have been caused by construction of the Gezende dam and little will be added by the Project construction.

Aquatic flora will tend to flourish but such change will be slow because the water is intact at present.

6.3.4 Risk resultant matrix

The foregoing considerations on the environmental impact of the Project were summarized in a risk resultant matrix as shown in Table 6.8. Entries in the matrix are possible/probable impacts of the Project activities on different aspects of the environment in different time periods broadly classified into preconstruction, construction and operation periods. The impact is expressed by the following indicators:

o: No effect expected

+: Positive effect expected

-: Negative effect expected

±: Neutral/mixed effect expected; there may be a change in the conditions of this aspect of the environment, but from an overall ecosystem perspective, such change can be judged neither totally beneficial nor totally harmful.

x: Undetermined; there is the possibility of some effect occurring, but with the present data available, it is impossible to predict the direction of the effect.

6.4 Laws

Environment-related articles are entered in the Constitution and other laws.

Article 3 of the Environment Law No. 18132 promulgated in 1983 provides the general principles regarding environment protection and prevention of environmental pollution. Sub-article c) among others states that "Institutions authorized to make decisions and evaluate projects concerning the use of land and resources shall pursue the objective of protecting and not polluting the environment, at the same time taking care not to affect development efforts adversely.

In and after 1983, some relevant laws and decree-laws were promulgated. These are; the Law on Protection of

Cultural and National Assets No. 18113, the National Park Law No. 18132, the Bosphorus Law, the Decree-Law Concerning the Establishment and Functions of the Environmental General Directorate, No. 18435, and the Building Code (Official Gazette No. 18749).

The Aquatic Products Law No. 13799 promulgated in 1971 contains provisions directly related to the dam:

> **Measures to be taken in dams and artificial lakes**
> **Article 8:** Before any water is introduced into dam reservoirs or other artificial lakes, an application must be made by the concerned parties to the Ministry of Agriculture to determine the measures that need to be taken regarding aquatic products, and any such measures indicated as necessary by the Ministry must be taken.
>
> **Measures to protect aquatic products from damage**
> **Article 9:** When inland waters are used for purposes such as irrigation or the production of energy, it is a condition that the measures necessary for protecting the life, propagation, conservation and production of the aquatic products existing in such waters be taken by the parties concerned. The Ministry of Agriculture shall determine what these measures should consist of.

6.5 Countermeasures

6.5.1 Decision-making process

There may arise objections from local residents concerning land expropriation, arrangement of structures, transportation and other environmental issues.

A sustainable development is the principle applicable to the environmental program in any country, but environmental quality involves so many aspects that it cannot be simplified into a single parameter like monetary value in economic analysis and, furthermore, the environmental issues in a country or region are usually argued in relation to social, economic and cultural conditions particular to the nation or the region. The decision-making process should, therefore, involve consultation with affected parties from the earliest possible stage to build up consensus.

6.5.2 Resettlement programs

The land expropriation for the Project is expected to be successfully completed by means of a reasonable amount of compensation to the affected people, but some land tenure problem may emerge. In this connection, a resettlement program should be worked out on the principle that the previous quality of life and amenity should be maintained.

6.5.3 Environmental considerations in design of project facilities

The proposed design of the Project involves some basic ideas to minimize adverse effects to the environment. For example, structures such as the power station and most parts of the waterway will be located underground. This kind of consideration should be made also at the stages of detailed design and construction, such as determining the alignment of access roads, selecting the area for construction plant and facilities, and installing the aggregates plant. These considerations should cover the preservation of landscape, control of air, water and soil pollution, noise control, safety, and communication with public.

6.5.4 Legal and institutional measures

The creation of large still water body risks water pollution and eutrophication which may be caused by nitrogen, phosphorus and heavy metals entering the reservoir as a result of increased human activities upstream. There are a few crucial measures to prevent these problems occurring, a legal restriction on disposal of unfavorable material in the reservoir or river leading into it with a heavy penalty for infringement and integrated regional plans to maintain human activities within appropriate limits.

6.6 Further Study

The main environmental issues were identified and countermeasures to them were proposed based on investigations to date. As in any reservoir project, it is important to have detailed social and environmental studies made so that future problems can be avoided by timely mitigation or avoidance. These studies should include a biological inventory study, fishery and tourism potential studies, and consultations, including questionnaire studies, with local people.

CHAPTER 7. PLAN FORMULATION

7.1 Background for the Plan Formulation

Of the electric energy produced in 1988, hydropower shared 60 per cent, followed by lignite-fired thermal power with 25 per cent. Lignite-fired thermal plants had been intensively developed through the 1980s, but its energy share has started to decrease after reaching a peak of 47 per cent in 1986 along with the commissioning of additional units at Karakaya and Altinkaya hydro plants. The exploitable reserve of lignite in Turkey is limited, and development of imported-coal thermal plants is being planned to meet the growing demand.

Turkey has been suffering from air pollution problem especially in the winter season, when most of households use lignite for house heating. The Government is making every effort to alleviate the pollution issues by such means as shifting of lignite to imported coal or imported natural gas, and further implementation of hydropower projects.

Of the economic hydropower potential of 121 billion kWh indigenous to Turkey, 20 per cent have been developed, 13 per cent are under construction. A continuous and steady development of the rest of 67 per cent is of vital importance to support the growth of economic activities in Turkey.

Such being the background, the Project was put into the stage of feasibility study as it is blessed with the following natural and socio-economic conditions:

(1) The Ermenek river has a relatively stable river flow fed by many springs located in the basin. These

springs are recharged by snow-melt mainly on the southern boundary of Ermenek river basin.

(2) The Ermenek river has an average slope of 1/72 in the downstream reaches of the prospective dam site. On the other hand, there is a wide river valley spreading towards upstream of the dam site. These topographic features are favorable for a dam and waterway type power development.

(3) The Project area is located close to regional demand centers and has, therefore, operational advantages such as less transmission losses in peak power supply.

(4) The Gezende hydroelectric power project is under construction downstream of the proposed dam site. Because of its limited capacity of the reservoir, it will function like a run-of-river type power plant. The firm energy would be greatly enhanced upon implementation of the Project. This firming-up effect would also be expected on the Kayraktepe hydroelectric power project planned on the main stream of the Göksu river.

(5) The reservoir area is sparsely populated because most of the houses in the area are located around springs, which are situated at higher elevations than the reservoir level conceived. Adverse effects of the Project on the people and farmland in the reservoir area will relatively be small.

7.2 Work Flow of Plan Formulation Study

Based on the results of Additional Detailed Investigations, it is judged that the lower part of the limestone at I-C dam site (in the elevation range below the groundwater level) is less karstic than the upper part, and that the water leakage through the limestone could be stopped by a grout curtain. Accordingly, the second best alternative of a rockfill dam at I-B site was ruled out at this stage of the study.

Also the bottom elevation of the limestone block, wherein the underground power house is planned, has been sounded through the drilling of hole Nos. SK-102, 106 and 108b. It is judged that the underground power house can be placed within the limestone block.

Accordingly, the tentative conclusion of the preliminary studies described in Section 4.3 has been confirmed. The best prospective development plan for the Project is a 1-step development with an arch dam at the I-C site, an underground power house, and the Erik Diversion Scheme.

The plan formulation study was carried out based on this basic development plan. The study procedure was as described below (Fig. 7.1):

(1) Assumptions and preparatory studies: (A) planning of the initial filling of the Ermenek reservoir; (B) assessment of risks accompanying unidentified water leakage paths in the limestone block of dam site; (C) assessment of power outputs and benefits; (D) construction cost estimation and planning.

(2) Studies of the more detailed development plan of the Project: (A) the optimum dam axis in the I-C site; (B) Erik Diversion Scheme; (C) underground power house; (D) addition of Erik power station.

(3) Optimization of the principal Project components such as type and capacity of spillway, drawdown of reservoir, diameter of power waterway, route of headrace tunnel, location of tailrace outlet, discharge capacity and type of the Erik Diversion Scheme.

(4) Selection of the optimum development scale

The development scale was represented in this study by the HWL of the Ermenek reservoir. The HWL was selected in place of the dam height or dam crest elevation as the HWL can be an independent parameter both from the extra dam height above HWL and from depth of the foundation excavation. The firm and secondary energies are dependent on HWL. The installed capacity is, in this study, dependent on the firm energy (see Sub-section 3.5.2). Thus, the development scale can be represented by a single parameter of HWL.

7.3 Assumptions and Preparatory Studies

7.3.1 Initial filling plan of reservoir

The optimum height of the Ermenek dam will be governed also by the initial filling period of the Ermenek reservoir, since it will have a large capacity (3,529 MCM in the case of a HWL of 675 m). This size of reservoir will be fully filled in about 2.4 years if the mean annual inflow of 1,466 MCM or 46.5 m^3/s including the Erik flow continues.

During the initial filling period, the output of Gezende power station will inevitably be decreased or stopped. The Ermenek power station can start its full power generation only after completion of the initial filling. In order to reflect this adverse effect to the plan formulation study, an initial reservoir filling plan was prepared based on the following assumptions and filling procedure:

(1) It is assumed that the average annual inflow will continue throughout the filling period.

(2) The first stage filling to fill the reservoir up to LWL will be started when construction works of such structures are completed as dam and curtain grout up to LWL;

power intake; bottom outlets and a tunnel spillway.

(3) Throughout the first stage filling period, necessary discharge will be released from the Ermenek dam to secure the originally planned firm outputs of Gezende power station.

(4) If the reservoir water level reaches LWL before completion of the dam construction works, the reservoir water level will be controlled to keep an adequate level by operating the bottom outlets and or tunnel spillway (the second stage filling).

(5) When the construction works of dam and curtain grout are completed, the third stage filling will be started, by generating power at 50 per cent of the firm energy.

(6) The third stage filling will be continued up to when the reservoir is filled for half the effective capacity. Thereafter, the full operation will be started as planned.

The filling period is shown for each alternative HWL in Fig. 7.3. In the case of HWL of 675 m, the reservoir filling period would be 24.8 months; the average rising speed of reservoir water level would be 0.31 m per day up to LWL, and 0.16 m per day from LWL to the end of the initial filling (see Fig. 7.4).

7.3.2 Risk of water leakage

In general, a geological investigation of the karstic limestone for dam construction will not be completed by the feasibility study. The investigation will be continued in a succeeding detailed investigation and during the construction period. The objectives of grouting tunnels and holes are not only to perform grout injection but also to search the whole limestone for all the potential leakage paths and karstic cavities. The investigation will finally be completed upon successful completion of the initial filling of reservoir.

There may be more or less unidentified water paths and karstic cavities in the limestone block. The risk of water leakage would become high, probably in proportion to the hydraulic head and the necessary area of grout curtain. The higher dam would have the higher risk of water leakage. To take into consideration this risk, the water leakage through the limestone block was assumed as described below:

Most dams constructed on limestone foundations in the world have more or less water leakage through foundations as shown below:

<u>Water Leakage Through Limestone Foundations</u>

Dam	Dam	Initial Stage		Final Stage	
Name	Height (m)	H. Head (m)	Leakage (cc/s/m2)	H. Head (m)	Leakage (cc/s/m2)
Chicoasen	262	252	0.8	no supplemental curtain	
Keban	211	202	130	204	15
Oymapinar	185	183	1	no supplemental curtain	
Kremasta	160	145	50		
Cannelles	151	85	200	138	0.4
Pueblo Viejo	140	135	0.04	135	0.03
Dokan	120	80	14	118	0

Source: Dam foundations on karstic formations, 1985, Commission Internationale Des Grandes Barrages

With reference to these leakage records, a basic rate of leakage was assumed at 1.0 $cm^3/s/m^2$ under the hydraulic head of 100 m. The water leakage for a case of HWL at 675 m was then assumed at 1.0 m^3/s.

7.3.3 Operation study and power benefits

Reservoir operation studies were carried out for each alternative plan to assess the energy outputs. Energy outputs of power stations located serially on the Ermenek river such as Nadire, Ermenek, downstream second at II-A or II-B site, Gezende power stations were estimated through simulation of the combined operation of serial reservoirs (see ANNEX-D for the operation rule).

In the simulation, a monthly inflow series of the 42 years from 1946 to 1987 was used. However, the series was rearranged into the following order so as to recover HWL at the end of the simulation period:

1982 to 1987 followed by 1946 to 1981

The reservoir evaporation loss rate was assumed at 70 per cent of the mean monthly pan evaporation records, or at a rate of 1,380 mm/yr. The water leakage loss described in the foregoing Sub-section 7.3.2 was deducted, in the simulation study, from the Ermenek reservoir and was returned to the Gezende reservoir.

As mentioned in Sub-section 3.5.2, the installed capacity of the Ermenek power station was determined with an annual capacity factor of 33 per cent on the basis of the firm energy output.

The firm energy was herein defined as the 100% dependable annual energy, which can be secured throughout the above 42 years' period of the inflow series. A 90% dependable power was obtained by serial method based on the 504 monthly energy outputs, which had been obtained through simulation for the 42 years inflow data.

Power benefits of the Project were assessed with the capacity value of US$121/kW, firm energy value of US¢.68/kWh, and secondary energy value of US¢2.33/kWh (Section 3.6). The capacity benefits were obtained based on the 90% dependable power. The firming-up benefits of the Gezende power station were assessed with the same criteria. The power benefits of Erik power station were assessed with the capacity value of US$159/kW only for firm power and the energy value of US¢2.33/kWh, assuming an alternative coal thermal power plant in view of its base operation as a run-of-river type plant.

7.3.4 Unit construction prices

The optimization studies were worked out on the basis of the construction method, time schedule and unit construction prices, which were examined and worked out for a prospective development scale of the Project; HWL at 675 m (see Chapter 9 for details).

7.3.5 Disbursement schedule and discount rate

Optimization studies were performed by comparing the capitalized net benefit of various alternatives based on the net benefit maximization criteria. The net benefit was obtained for each alternative in accordance with the procedure shown in Fig. 7.2. The net benefit was assessed by capitalizing the annual net benefit for the evaluation period of 60 years from the starting year of detailed design works, with a discount rate of 9.5 per cent. This discount rate was provided to the Study Team by EİE as the opportunity cost of capital (OCC) of the energy sector in Turkey.

The economic construction costs were disbursed in accordance with the following disbursement schedule, which had been prepared based on the construction time schedule given in Plate P8.

Year	Disbursement	Year	Disbursement
1	0.5 %	6	11.0
2	1.9	7	12.0
3	5.8	8	20.7
4	3.1	9	25.8
5	7.8	10	11.4

7.4 Study of Development Plan

7.4.1 Selection of dam axis in the I-C site

In the Görmel Gorge 3 alternative dam axes were proposed by the Study Team. These are I-Ca, I-Cb and I-Cc located in the order towards downstream (see Plate A34). The geological profiles of these dam axes are shown in Plates G5 to G7.

The dam axis I-Ca is located at the entrance of the Gorge. The access to the site is easiest among the 3 axes. The valley has a narrow V-shaped section for the elevation range below 590 m; but the slopes become gentle and the valley width becomes wide for the elevation range above 650 m, which is the shoulder elevation of the valley; with the transition elevation range between 590 m and 650 m. This dam axis is advantageous for a low height arch dam in view of its narrow profile and easy access.

The dam axis I-Cb is located about 80 m downstream of the I-Ca axis. The valley profile is similar to that at I-Ca except for the higher shoulder elevation at around 670 m. In this axis there is a Joint Jc-1 on the right bank (see

Plate G6). The rock mass located above the Joint should be removed when the dam is situated here. The access to this site will be mainly from the upstream side of the Gorge.

The dam axis I-Cc is located about 150 m downstream of the Fault F-2. The Gorge has the narrowest and highest V-shaped section around this axis, which resembles the Viont Dam in Italy. The riverbed elevation is about 497.5 m (Fig. 5.1). The valley shoulder elevation is about 720 m, which permits construction of a high dam exceeding 200 m in height from topographic point of view. The access to this dam site will be mainly from the downstream side of the Gorge.

Such access tunnels as constructed on the downstream side of the dam axis will permit permanent access even after impounding of the reservoir. Since all the 3 dam axes are situated in the limestone block, this permanent access is prerequisite to perform leakage measurement, inspection and supplemental grouting works after commencement of initial filling of the reservoir. To enable these works also in the case of 2 upstream dam axes I-Ca and I-Cb, some special access measures will be required in addition to the access tunnels for construction purposes.

As described in Section 5.3, it has been judged that all the 3 dam axes proposed in the I-C site, the Görmel Gorge, have the rock foundations where a high arch type dam can be constructed, and that water leakage through the karstic limestone block could technically be stopped by a grout curtain (see Plate P14 for the proposed grouting plan).

In order to select the dam axis by least cost criteria, a preliminary design of arch dam was worked out for the following 9 alternatives of dam axis and crest elevation:

For I-Ca dam axis: 640, 660, 670 m

For I-Cb dam axis: 640, 660, 670 m

For I-Cc dam axis: 660, 680, 700 m

The above dam crest elevations were selected based on such conclusion of the preliminary study as the optimum elevation would be in the range between 650 m and 700 m.

Each dam was designed as a parabolic type arch dam. Plan, crown cantilever section, and upstream and downstream developed profiles are shown in Plates A35 to A37 for the 9 alternatives. The dam volumes were estimated as summarized below:

Dam Volume (1,000 m^3)

Dam Crest Elevation	Dam Axis I-Ca	I-Cb	I-Cc
640 m	213	165	-
660	374	288	233
670	517	400	-
680	-	-	270
700	-	-	335

A dam height-dam volume curve, a dam height-dam excavation volume curve, and a dam height-net construction cost curve for dam excavation and dam concrete are shown in Fig. 7.6. The dam axis I-Cc will have the least figures all in the dam concrete, excavation and construction cost. I-Ca will have the largest concrete volume, while I-Cb will have the largest excavation volume.

The dam at I-Ca has the longest crest length, resulting in the largest concrete volume. The largest excavation volume at I-Cb axis is required on account of the thick weathered zone on the right bank (see Plate G6).

The unit cost of dam concrete was estimated at 130 US$/m^3 based on the quantities of 270,000 m^3 for a dam having a crest elevation at 680 m and located at I-Cc axis. The unit cost for dam excavation was estimated at US$10/m^3.

The cost difference amounts to US$27 million for a dam crest elevation at 670 m between the least cost axis I-Cc and the second least-cost axis I-Cb (Fig. 7.6). This difference amounts to 66 per cent of the cost of US$41 million for the I-Cc axis. Accordingly, the superiority of I-Cc axis will not change even including some other advantageous and disadvantageous factors to I-Cc axis compared to I-Ca and I-Cb, such as (1) some decrease of the length of headrace tunnel for I-Cc axis; (2) some increase of grout curtain area for I-Cc; (3) some increase in length of access tunnel to I-Cc dam site; (4) some decrease of unit costs for larger work volumes of I-Ca and I-Cb axes.

The above advantage of I-Cc is attributable to the narrow and steep valley topography and the rock conditions without thick and highly weathered zone (see Plate G7).

The alternative dam axes I-Ca and I-Cb were thus ruled out at this step of the study.

7.4.2 Viability of Erik Diversion Scheme

The Erik river has an average runoff of 3.5 m^3/s with a stable flow duration (see Fig. 7.11). By constructing a tunnel of 6 m^3/s in conveyance capacity, most of the Erik flow can be diverted to a headrace tunnel of the Project.

An intake weir site was first conceived on the Erik river at an altitude of about 700 m, from where the Erik flow could be diverted by a tunnel of about 1,500 m long. The weir site was, however, shifted to an upstream point near the Erik spring because a large scale active landslide had been found on the upstream reaches in the Additional Detailed Investigation Stage.

An economic comparison was made for the 2 cases; with and without the Erik Diversion Scheme. The comparison was made for the proposed development scale of the Project, that is, HWL of the Ermenek reservoir at 675 m but without the Erik power station of which economic viability will be examined in Sub-section 7.4.4.

The principal features and economic indices are as summarized below (see ANNEX-D for details):

No.	Description	Unit	Without Erik	With Erik
(1)	Installed capacity	MW	300	320
(2)	90% dependable power	MW	290	294
(3)	Annual energy	GWh		
	- firm		865	925
	- secondary		78	97
	total		943	1,022
(4)	Economic construction cost	mil.$	336.3	342.3
(5)	Annual equivalent cost	mil.$	33.0	33.6
(6)	Annual benefit excluding firm-up benefit of Gezende	mil.$	60.0	62.5
(7)	Annual net benefit	mil.$	27.0	28.9

The annual equivalent cost above was obtained in the same way as described in Sub-section 4.3.3

As shown in the table above, the Erik Diversion Scheme will increase the firm energy by 60 GWh, the secondary energy by 19 GWh, and the annual net benefit by US$1.9 million. It was concluded that the Erik Diversion Scheme should be included in the Project.

7.4.3 Underground power house

An economic comparison was made of the aboveground and underground types of penstock and power house for a case of HWL at 675 m. The tailwater level was assumed at 333 m for the underground type having a tailrace tunnel; at 337 m for the aboveground type without tailrace tunnel.

Plan and profile of the aboveground penstock are shown in Plate A29, and of the underground type in Plates P17 to P19. An economic comparison was made as summarized below:

No.	Description	Unit	Aboveground	Underground
(1)	Length of penstock	m	1,035	440
(2)	Installed capacity	MW	315	320
(3)	90% dependable power	MW	294	294
(4)	Annual energy	GWh		
	- firm		911	925
	- secondary		97	97
	total		1,008	1,022
(5)	Economic construction cost	mil.$	341.5	342.3
(6)	Annual equivalent	mil.$	42.3	42.4
(7)	Annual benefit excluding firm-up benefit of Gezende	mil.$	62.1	62.5
(8)	Annual net benefit	mil.$	19.8	20.1

The aboveground and underground plans are almost comparable to each other although the underground type yields a slightly higher net benefit. This is, however, mainly attributable to the additional head of 4 m available for the underground type.

The underground type has an operational advantage because of its shorter length of the pressure shaft: 1,035 m in horizontal length for the aboveground type; 440 m for the underground type. Also the underground type will not affect the natural environment on the route and will be free from slope protection works. Thus the underground pressure shaft and power station were adopted for the Project.

7.4.4 Addition of Erik power station

Because of the weir location near the Erik spring, the intake water level will be around 815 m, which is 140 m higher than the proposed HWL of the Ermenek reservoir (675 m). Therefore, the discharge of 6.0 m^3/s diverted will have a potential power of 8,200 kW above the HWL. Accordingly, a facility will be required to dissipate this energy before pouring into the main headrace tunnel. The function of this energy dissipater can be obtained also by installing a generating equipment.

The Erik power station, if added to the Erik Diversion Scheme, would have an installed capacity of 6.7 MW and annual energy output of 31.9 GWh. In this case the Erik water will be utilized for a total gross head of 487 m (145 m at Erik + 342 m at Ermenek).

The annual power benefit was assessed at US$1.39 million; the annual equivalent of construction cost at US$0.29 million; the annual net benefit at US$1.10 million. It is thus decided to add a power plant to the Erik Diversion Scheme.

7.5 Optimization Study of Project Components

(1) Discharge capacity of spillway

The larger spillway capacity reduces the flood storage capacity to be allocated above HWL, resulting in the lower dam height. In this case the spillway cost increases while the dam cost decreases; and vice versa.

A comparative study was made to obtain an economic combination of the flood storage capacity and the spillway discharge capacity with the least cost criteria in principle. However, the concept of a minimum capacity of flood discharging facilities of dam was introduced to avoid the selection of an extremely small capacity. The minimum capacity of flood discharging facilities from the dam was determined at 2,200 m^3/s, as the 200-year probable flood at the dam site. This flood frequency was selected in accordance with the standard for the spillway design discharge in Japan.

The spillway discharge capacity and flood water level for the case of HWL at 675 m were obtained as the least cost meeting the minimum discharge criteria. The optimum combination is as shown below (see ANNEX-D for details):

- Maximum outflow : 2,600 m^3/s
- Maximum flood storage depth above HWL : 3.3 m

(2) Drawdown of reservoir

A comparative study was made for the 5 alternative drawdowns of the reservoir. The study was made for the prospective HWL of 675 m. The principal features and economic indices are as summarized below:

No.	Description	Unit	Drawdown 30m	45m	60m	75m	90m
(1)	LWL	m	645	630	615	600	585
(2)	Effective reservoir capacity	MCM	1,329	1,879	2,339	2,715	3,009
(3)	Capacity-inflow ratio	%	91	128	160	185	205

No.	Description	Unit	Drawdown 30m	45m	60m	75m	90m
(4)	Mean reservoir water level of 42 years	m	668	664	660	648	644
(5)	Installed capacity	MW	273	297	320	328	331
(6)	90% dependable power	MW	260	276	294	274	272
(7)	Annual energy	GWh					
	- firm		825	892	925	950	952
	- secondary		179	127	97	52	43
	total		1,003	1,018	1,022	1,002	995
(8)	Economic construction cost	mil.$	322.0	335.8	342.1	348.6	351.8
(9)	Annual cost	mil.$	40.5	42.3	43.1	43.9	44.3
(10)	Annual benefit including firm-up benefit	mil.$	73.3	76.1	78.8	76.9	75.6
(11)	Annual net benefit	mil.$	32.7	33.8	35.7	33.0	31.3

As shown in the table above and in Fig. 7.7, the annual benefit approaches a peak at the drawdown of 60 m. The reservoir drawdown was then determined at 60 m (see ANNEX-D for details).

(3) <u>Diameter of headrace tunnel</u>

An empirical formula was derived by F. Fahlbusch based on 394 examples in the world, to determine the diameter of a power tunnel (Water Power & Dam Construction, February 1987). The formula is given by:

$$D_c = 0.62\ Q^{0.48}$$

In the case of the design discharge of $Q = 116.6\ m^3/s$ for a HWL of 675 m, this formula gives a diameter of 6.1 m. This diameter yields a hydraulic gradient of

1/747 at n = 0.012, which is considered reasonable for a pressure power tunnel.

Further to examine the applicability of the above formula, an economic diameter of the headrace tunnel was obtained with the least cost criteria, taking into consideration the power loss due to the hydraulic loss of head in the headrace tunnel. The least cost diameter for the discharge of 116.6 m^3/s was obtained at 6.30 m for coefficient of roughness of n=0.012. However, the cost differences were as small as less than US$0.1 million per annum for the diameter range from 6.0 m to 6.6 m, in which the above diameter of 6.1 m falls. Accordingly, it is judged that the above empirical formula can be applied to the Project.

(4) Route of headrace tunnel

A comparative study was carried out on 5 alternative routes of headrace tunnel shown in Plates A30 to A32. The alternative route A has a straight alignment between the intake and surge tank sites. Because of the topography along the route, it incurs long work adits amounting to 3,360 m in total.

The economic route was obtained with the least cost criteria of a sum of the construction costs of work adits and headrace tunnel and the power loss due to the hydraulic loss of head in the headrace tunnel. The alternative routes were selected so that they always pass the mountain side of the contour line corresponding to HWL.

The annual costs were obtained as shown in Fig. 7.8. From the figure the Route Bb was selected as the least cost among the 5 alternatives.

(5) Diameter of pressure shaft (penstocks)

The economic diameter of steel liner which will be embedded in the pressure shaft was studied for the 2 cases: single lane and two lanes.

(A) Single lane shaft

F. Fahlbusch also derived another empirical formula to determine a diameter of steel liner embedded in the pressure shaft. The formula is given by:

$$D_s = 1.12\ H^{-0.12}\ Q^{0.45}$$

The above formula gives a diameter of 4.9 m for a design discharge of 116.6 m^3/s. In the same way as for the economic diameter of headrace tunnel, it is judged that the above empirical formula can be applied to the Project (see ANNEX-D for details).

(B) Two lane shaft

The diameter of two lane steel liner was determined to be 3.6 m in the same way as for the single lane (see ANNEX-D for details).

(C) Selected lane number and diameter

An economic comparison was made between the single lane and two lane alternatives as summarized below:

Items	Unit	Single Lane	Two Lanes
1. diameter of steel liner	m	4.9	3.6
2. tunnel cost	Mil.$	1.43	1.99
3. steel liner cost	Mil.$	15.81	14.95
4. total cost	Mil.$	17.24	16.94
5. annual equivalent of construction cost	Mil.$	1.90	1.86
6. annual power loss	Mil.$	0.33	0.42
7. annual total	Mil.$	2.22	2.28

As shown in the table above, the single lane has the higher construction cost, lower power loss, and lower annual total cost compared to the two lanes. However, the difference is as small as US$0.06 per annum.

In the case of single lane, the maximum excavation diameter will be 5.9 m including working clearance of 0.3 m and extra excavation of 0.2 m. The vertical height becomes 8.3 m in an inclined shaft of 45 degree, and some difficulties are expected in construction works. In the case of two lane shaft, it becomes 6.5 m; and the shaft excavation equipment can be used for the double length compared to the single lane, resulting in a higher depreciation of the excavation equipment.

Accordingly, the two lane shaft was adopted in the Project, and its diameter was determined at 3.6 m.

(6) <u>Location of outlet of tailrace tunnel</u>

A comparative study was carried out on the 4 alternative routes and outlet locations of the tailrace tunnel as shown in Plate A33. The river profile around the outlets is also shown in Fig. 7.9. As shown in this figure, there are rapids of about 2 m high and the Route B is located at an immediate downstream point of this rapids.

Principal features and economic indices are summarized below for the alternative routes compared to those of Route A (see ANNEX-D for details):

	Description	Unit	Alternative No. A	B	C	D
(1)	Incremental length of tailrace tunnel	m	0	+513	+638	+1,098
(2)	Maximum head increased	m	-	+7.0	+8.0	+10.0
(3)	Incremental annual cost	mil.$	-	0.29	0.36	0.63
(4)	Incremental annual benefit	mil.$	-	0.76	0.77	0.78
(5)	Incremental annual net benefit	mil.$	-	0.35	0.26	-0.08

As shown in the above table and in Fig. 7.10, the annual benefit increases sharply from the alternative Route A to Route B owing to the rapids, but is stagnant in the far downstream points. On the other hand, annual equivalent cost and annual power loss continues increase along with the length of the tailrace tunnel. Thus the Route B becomes the least cost among the 4 alternatives. Accordingly the Route B was selected.

(7) Design discharge of Erik Diversion Scheme

A flow duration curve was developed based on the data of 3 hydrological years (October through September): 1965/1966, 1969/1970, 1970/1971. The duration curve is shown in Fig. 7.11.

A free flow tunnel of the minimum working dimension shown in Plate P22 has a discharge capacity of 6.0 m^3/s for a coefficient of roughness of 0.014 and a hydraulic gradient of 1/1,000. This discharge capacity corresponds to 4 per cent in the probability of exceedence on the flow duration curve. It means that there will be little overflow from the Erik diversion weir. There are no farmland and houses which are dependent on the Erik flow. The design discharge of the diversion tunnel was then determined at 6.0 m^3/s.

(8) Type of Erik diversion tunnel

As the type of the Erik diversion tunnel, a free flow tunnel type and a pressure flow tunnel type are conceivable. Of these, the free flow type was found cheaper as shown below:

(million US$)

Work Items	Free Flow Tunnel	Pressure Tunnel
Tunnel excavation	1.64	1.76
Lining concrete	1.29	1.57
Consolidation grout	-	0.38
Total	2.93	3.71

Moreover, the free flow type with semi horse-shoe type is preferable for tunneling works of a small diameter. As a result, the free flow type was selected (see

ANNEX-D for details).

(9) Addition of an excess water spillway to Erik headtank

If an excess water spillway is not provided to the Erik headtank, the Ermenek power station will lose the Erik flow during inspection and maintenance of the Erik power station. This loss was estimated at about US$57 thousand per annum on the basis of an assumption of a station shut-down period of 7 days per annum for regular inspection, and once for 30 days in 7 years for overhaul.

If a surface type spillway is added, the Ermenek power station can utilize the Erik flow even during these periods. An annual equivalent of the construction cost of this spillway was estimated at about US$30 thousand being lower than the benefit of US$57 thousand. The excess water spillway is thus added to the Erik waterway system.

(10) Tailwater level of Erik power station

A tailwater level of the Erik power station is preferable to be set at or above the HWL of Ermenek reservoir, in such views as (A) an adverse effect of deep setting to a small scale turbine when it is set below HWL; (B) a difficulty to make the tailrace dry for inspection.

During surging in the Ermenek headrace tunnel, the water level in the Erik tailrace will be affected. The maximum water level is about 18 m above the HWL of Ermenek reservoir. When the tailwater level is set at the HWL of Ermenek reservoir, a height of tailrace chamber would be about 25 m. This costs about US$0.2 million, but it contributes to power generation at the Erik power station by lowering the normal tailwater level. A tailwater level raising by 10 m decreases annual power benefit by about US$1.2 million. Thus the normal tailwater level was set at the lowest level in a range preferable, or at the HWL of Ermenek reservoir.

7.6 Optimum Development Scale

A comparative study of HWL was performed for 11 cases from 645 m to 695 m at intervals of 5 m. Based on the preliminary designs of the principal Project components as described in the foregoing sections, the Project benefit and cost were estimated. These are shown in Figs. 7.12 and 7.13 with the alternative HWL as abscissa.

The benefit capitalized with a discount rate of 9.5 per cent reaches a peak at a HWL of 675 m; and starts to gradually decrease thereafter. This is due to the adverse effect of the initial filling. The capitalized cost increases almost in proportion to the HWL. The capitalized net benefit curve shows that the HWL of 675 m yields the maximum net benefit.

The development scale of the Project was thus determined as HWL=675 m. The installed capacity was determined at 320 MW based on the firm energy of 925 GWh with an annual capacity factor of 33 per cent. Although the firm energy will be secured for the 42 years' inflow records from 1946 to 1987, the 90% dependable output will be 294 MW, or 92 per cent of the installed capacity due to such reservoir water levels as go down beyond the rated water level in the driest years.

7.7 Superiority to 2-step Development

As described in Sub-section 4.3.3, an economic advantage of 1-step development has been proved for an alternative HWL of 645 m. This economic superiority will not change even for the proposed HWL of 675 m as described below:

(1) The power outputs and benefits will increase almost in the same order both in the 1-step development and the 2-step developments with a downstream dam.

(2) The incremental costs of 1-step development will be larger by about US$6 million than that of 2-step development due to the change of tunnel diameter from 5.6 m to 6.1 m. However, this difference is negligibly small compared to the economic advantage of US$56 million shown in Sub-section 4.3.3.

(3) Accordingly, the economic advantage of 1-step development will not change.

CHAPTER 8. DESCRIPTION OF PROJECT FACILITIES

8.1 River Diversion Works

In view of the dam type of thin concrete arch, the diversion design flood was determined to have a peak discharge of 900 m^3/s as a 5-yr probable flood. A design flood hydrograph was constructed on the basis of that recorded at Station 1714 (Kayraktepe dam site) on December 11 to 12, 1971, which had a peak discharge of 940 m^3/s and had a voluminous shape compared to the other floods observed.

The river diversion works will consist of one diversion tunnel, upstream and downstream coffer dams. The diversion tunnel will be about 365 m long and will have a section of horse-shoe type with 2R=7.0 m (see Plate P12). It will be constructed in the right abutment. Both the upstream and downstream coffer dams includes primary coffer dams of earthfill type.

When the diversion design flood occurs, the maximum outflow through the diversion tunnel will be 650 m^3/s; the maximum upstream water level 534.3 m; the maximum river water level at the tunnel outlet 515.9 m (see Fig. 8.2). Based on these the crest elevation and height above riverbed of these coffer dams are determined as summarized below:

No.	Coffer Dam	Dam Type	Crest Elevation	Dam Height
1.	Upstream primary	earth	EL. 510 m	12.5 m
2.	Upstream main	concrete arch	535	37.5
3.	Downstream primary	earth	505	7.5
4.	Downstream main	concrete gravity	516	18.5

The downstream main coffer dam will be used also as the sub-dam after completion of all the construction works. It will create a plunge pool for dissipation of the energy of falling discharge released from the bottom outlets and or crest overflow spillway.

8.2 Dam

(1) Dam features and geometry

The Ermenek dam will be of thin concrete arch type. The dam features are as summarized below (Plate P10):

(A)	Dam crest elevation		:	680.0 m
(B)	Design flood water level	(DFWL)	:	678.3 m
(C)	Normal high water level	(HWL)	:	675.0 m
(D)	Low water level	(LWL)	:	615.0 m
(E)	Foundation elevation		:	490.0 m
(F)	Freeboard above DFWL		:	1.7 m
(G)	Drawdown		:	60.0 m
(H)	Dam height		:	190.0 m
(I)	Crest length		:	165.8 m
(J)	Ratio (H)/(I)		:	0.87
(K)	Dam volume (net)		:	270,000 m^3

The Ermenek dam was provisionally designed aiming at an estimate of the necessary volumes of dam concrete and excavation of abutments for the construction cost estimation and planning.

The dam geometry was designed as a parabolic type, which had been developed and applied to 10 dams in Japan since 1969. Radius of curvature varies from arch crown towards abutments in this type. The radius is the shortest at the arch crown, and the largest at the abutment. The main feature of this type is that the central angle can be reduced up to around 75 degrees so far constructed in Japan. This feature contributes to stability of the abutment by directing the arch thrust deep into the abutment, while maintaining good stress conditions in the central part around the arch crown.

However, because of the very narrow width of the valley bottom, the central angle was provisionally set at 110 degree at this level of design to attain a good arch action also for arch slices near foundation, where arch rise becomes small while dam thickness becomes large.

The dam stresses were preliminarily analyzed by trial load method both for the normal and earthquake conditions. The design earthquake was determined at K_h = 0.10 as 2 times the maximum credible earthquake of the site in view of the dam type of thin concrete arch. The horizontal and vertical stresses are shown in Plate P11 for the 2 conditions. The stresses are almost within the allowable stresses, which are given below for the normal conditions and are increased by 20 per cent for the earthquake condition:

	Allowable Stresses
<u>General</u>	
- Compression	70 kgf/cm^2
- Tension	-7
<u>Specific</u> 1/	
- Vertical tension on upstream surface near bottom	-30
- Vertical compression on upstream surface near bottom, and horizontal compression on downstream surface near bottom	50

1/: adjusted allowable stresses only for the purpose to judge the results obtained by the trial load analysis, which incurs some specific calculation errors.

In Plate P11, the minimum fresh rock lines necessary for the abutment to resist the arch thrust are also shown. Since all the arcs extending from the upstream end of the abutment are on the mountain side from the assumed fresh rock line of the valley, it is judged that the dam abutments have been excavated deep enough to secure the stability of the abutment. The much excavation for the arch slices in the upper elevations was designed to attain the steepest slope of the abutment at 1:0.3 for the slope protection works before placement of the dam concrete.

The above dam design needs a thorough review in the detailed design stage especially in (A) the reduction of the central angle and dam thickness at lower elevations; (B) the steepest excavation slope of the abutments.

(2) Dam design flood

A Probable Maximum Flood (PMF) was originally derived by EIE. The PMF was reviewed and revised to two new PMFs: PMF-1 for January, PMF-2 for February (see Section 5.5). These are characterized as shown below:

Description	Unit	Original	PMF-1	PMF-2
1. Peak discharge	m^3/s	3,700	5,900	5,400
2. Base flow incl. snow-melt	m^3/s	1,300	100	1,300

(3) Dam design freeboard

The dam crest elevation was determined with a design freeboard of 1.7 m provided above the design flood water level. This freeboard was adopted on the basis of the following considerations:

The maximum wave height was estimated to be about 1.6 m by the SMB method for a wind speed of 20 m/s on the 10 minutes average. This wind speed was assumed with reference to the maximum instantaneous wind speed of 23.2 m/s observed at Mut in the 21 years from 1966 to 1986. The prevailing wind direction was NW.

The Oymapinar arch dam located to the west of Ermenek basin has a freeboard of 1.4 m above the full supply level, and 2.4 m including the parapet height.

8.3 Spillway

(1) Type of spillway

In view of the dam construction in the V-shaped valley having a narrow width of 5 to 20 m at the river water level, a combination of the bottom outlets, a tunnel spillway and non-gated overflow crests was adopted.

If the tunnel spillway is excluded from the planned spillway system, a discharge capacity of about 1,600 m^3/s will be necessary for the overflow spillway to attain the design flood water level of 678.3 m jointly with the bottom outlets. To release this amount of discharge from an overflow crest of 40 m long for example, the operating head of about 7.4 m will be necessary, and a unit discharge of 40 m^3/s will flow down over one meter of the overflow crest. This large unit discharge may endanger a high and thin arch type dam. If the length of overflow crest is increased, this unit discharge will decrease but much of the overflowing jet will fall on the valley walls of both sides and will rush down the wall surface towards the valley bottom. Such a flow condition was judged not suitable in view of the wall slope protection. The overflow type was thus selected as the secondary spillway with a limited crest length.

(2) <u>The spillway system</u>

Two bottom outlets will be provided in the dam body. The outlet gates will have the dimension of 2.5 m wide and 4.0 m high, and the center elevation of 545 m. The discharge capacity will be 670 m^3/s at LWL, and 940 m^3/s at HWL. The bottom outlets will be operated (A) for avoiding a rise of the reservoir water level or for lowering it when water leakage is found during the initial filling of reservoir; (B) for discharging the flood inflow together with the other spillway components; (C) for releasing the water towards downstream when it is required in addition to the discharge through power station.

It is recommended that a tunnel spillway be adopted as the main discharge facility of the spillway system in order (A) to discharge the flood water far downstream of the dam foundation; (B) to enable, together with bottom outlets, an early commencement of the initial reservoir filling, without awaiting completion of the dam construction works.

The spillway tunnel will be 263 m long, and will have a circular section of 9.0 m in diameter and an invert elevation of the inlet at 630.42 m (see Plate P13). At 73 m downstream of the inlet, 2 pressure gates will be provided. The gates will be 3.0 m wide and 7.0 m high with their center elevation at 630 m.

A non-gated overflow crest of 40 m long will be provided on the dam crest as the normal overflow spillway.

The overflow crest elevation will be set at 675 m being equal to HWL. This will have a discharge capacity of 500 m^3/s at an operating head of 3.3 m. The overflow crest length of 40 m was selected in view of the narrow width of the valley bottom.

In addition to the above spillway system, two emergency overflow crests of 30 m long each will be provided on the dam crest, one to the right of the normal overflow crest and the other to the left. The crest elevation will be set at 678 m being 3 m above the HWL, in order to limit the usual overflows to the normal overflow crest. These emergency overflow crests will be provided to cope with such an incident as malfunctioning of one of the 4 gates of the tunnel spillway and bottom outlets.

The total crest length of the normal and emergency overflow spillways will become 100 m. This total length was selected in conjunction with the dam crest length of 165.8 m.

(3) Flood operation rule

Since the Project purpose is only the hydropower generation, the flood inflow will be released in accordance with the inflow = outflow rule in principle, or the gates will be operated to keep HWL as far as possible.

(4) The design flood water level

The design flood water level was determined at 678.3 m by routing the 2 PMF assuming an initial reservoir water level at HWL. Such a serious flood condition as the reservoir water level rises beyond EL. 676 m would continue for about 2.5 days (see ANNEX-D for details).

8.4 Grout Curtain

The grout curtain works will be the key of this Project. As shown in Plate P14, the grout curtain will be provided to all the vertical section of the limestone block down to an elevation of 400 m in general. The curtain will be extended further down to the elevation of 350 m in the right bank end for about 540 m in horizontal length. In the left bank, the grout curtain will horizontally be connected to the Ophiolitic rocks of the Ermenek Ophiolitic Melange; and in the right bank to the marl of the Görmel Formation. In the left bank, limestone becomes thinner towards upstream. Since rocks below the limestone block are low pervious ophiolitic, the bottom of the grout curtain will be raised along the bottom surface of the limestone block (Plate G12).

The grout curtain works will consist of construction of grouting tunnels of 13,580 m in total length, grout hole drilling of 386,000 m in total depth, and grouting of about 39,000 ton in cement weight. The objectives of the grout curtain works are not only to perform the grouting but also to investigate and search the limestone for karstic cavities and potential water leakage paths through the tunnel excavation works and grout hole drilling works.

In view of the different degree of karst by place in the whole grouting area and giving special weight to the zone around the dam, the following 3 patterns of grouting are planned:

Pattern 1: in 2 lanes at an interval of 2 m (average grout hole density is 1.00 m/m^2 of grouting area)

Pattern 2: in one lane at an interval of 2 m (average density: 0.50 m/m^2)

Pattern 3: in one lane with primary holes at an interval of 4 m and secondary holes at an intermediate point for two thirds of the area (average density: 0.25 m x 1/3 + 0.50 m x 2/3 = 0.42 m/m^2)

Pattern 1 will be applied to all the area within 300 m from the dam in horizontal length. The grouting of this pattern will be performed until a target Lugeon value of less than 1.0 is attained.

The remaining area on the left bank will be grouted by Pattern 2. Of the remaining area in the right bank, the upper part above EL. 510 m, where the limestone is karstic, will be grouted by Pattern 2, while the rest below EL. 510 m will be grouted by Pattern 3. In the area of Pattern 3, the secondary grout holes will be drilled mainly at the right end part where the downstream end of the curtain is planned at EL. 350 m. In Patterns 2 and 3, the grouting purpose is to fill up such solution openings as would cause serious water leakage after impounding of the reservoir. The target Lugeon value of these patterns is set at 10.

A drainage system will be provided on the downstream side of the grout curtain to reduce the uplift pressure to the dam body and abutment foundation. These drain holes will also be used for monitoring water leakage. The drainage holes will be drilled mostly from the grouting tunnels. The horizontal interval of drainage holes will be about 10 m in the area around the dam; and 10-20 m in the other areas.

The grouting tunnels of 3.5 m in internal diameter will be excavated at 6 elevations in the left bank with standard vertical intervals of 40 m. The lowest tunnel will be at 465 m at the invert elevation. The clearance between the dam foundation and the tunnel crown will be 21 m. In the right bank, another tunnel will be added to an elevation of

425 m to facilitate the grouting works planned for the right end. All the grouting tunnels will be concrete lined except for GTL-6 and GTR-6 at the top.

In order to facilitate access to the grouting tunnel, 5 access tunnels will be excavated on the downstream side of the Gorge. The tunnel will have a semi-horseshoe section with a height of 2.5 m and a width of 2.6 m, and will be used for railway transportation.

In the right bank 5 access tunnels will be excavated. Of these, the upper 2 will be excavated on the upstream side and will be closed before filling of the reservoir. These tunnels are planned aiming (1) to avoid intersection with the headrace tunnel and the spillway tunnel; (2) shorter tunnel lengths compared to a plan on the downstream side.

The lower 3 access tunnels will be excavated on the downstream side of the Gorge.

Three vertical elevator shafts will be provided: No. 1 on the left abutment of the dam; No. 2 about 180 m to the south from the right abutment; No. 3 about 1,160 m to the south from the second. These shafts will have the following dimensions:

Shaft No.	Diameter (m)	Height (m)
No. 1	4.0	215
No. 2	4.0	275
No. 3	6.0	255

Shaft Nos. 1 and 2 will be used for movement of the workers, transportation of light materials, and to provide space for air duct and other pipes. Shaft No. 3 will be mainly used for excavation, lining, drilling and grouting works in the lowest grout tunnel GTR-0. After completion of the construction works, these shafts will be used for inspection and maintenance works.

8.5 Waterway

The waterway will consist of an intake, a headrace tunnel, a headrace surge tank, underground pressure shafts in double lanes, a tailrace surge tank, a tailrace tunnel, and a tailrace outlet (see Plate P15).

(1) Power intake

The power intake will be of shaft type. The invert elevation of the inlet will be set at 600 m being 15 m below LWL. A trash rack will be placed but without raking equipment in view of the large drawdown of the reservoir being 60 m, and a high location of the inlet being 100 m above the riverbed. The gate shaft will be 10 m in excavation diameter and 80 m in height. Stop logs will be provided to facilitate maintenance works of the gate (Plate P16).

(2) Headrace tunnel

The headrace tunnel will be of pressure type and will have a circular section of 6.1 m in internal diameter. The tunnel length will be about 9,042 m. The average slope of the tunnel will be 1/396. The tunnel will be concrete lined for the whole section. The design lining thickness will be 0.40 m, while the extra excavation will be set at 0.20 m outside the lining line. In the work quantity and cost estimate, excavation and concrete volumes for this extra excavation were also taken into account.

A rock supporting system for tunnel excavation was planned to be either by NATM (the New Austrian Tunneling Method) or using steel supports depending on rock conditions. No support or a light supporting system would be sufficient for the limestone sections. A heavy support system would be required for the matrix layer sections.

(3) Headrace surge tank

The headrace surge tank will be of chamber type with a riser shaft. This type was selected in view of a high head of 342 m in maximum gross and a large drawdown of 60 m. The diameter of riser shaft will be set at 11 m to have a free surface area necessary for securing the stability of water surface under normal operation.

Sections of upper and lower chambers and surging water levels are shown in Plate P17. The upper chamber will have an overflow wall of 5 m high at the boundary with the riser shaft. The other end of the upper chamber will be connected to a ventilation tunnel, which will be used as an access tunnel during construction.

Surging wave curves are shown in Fig. 8.3.

No surge tank gate will be provided. Inspection and maintenance works of the waterway will be performed by completely shutting down the operation of the Ermenek and Erik power stations.

(4) Pressure shafts (penstock)

The pressure shaft will be branched into 2 lanes at a point 60 m downstream of the surge tank. The horizontal length of the pressure shafts will be 488 m. The center elevation of the shafts will be 580 m at the upstream end and 320 m at the downstream end. The shaft will be inclined at 45 degree, and the length of inclined portion will be 368 m long. The diameter will be 4.9 m in the single lane part and 3.6 m in the two lane part. An inlet valve will be provided at the downstream end of each shaft.

(5) Tailrace surge tank

Since the length of tailrace tunnel will amount to 1,764 m, a surge tank will be provided 80 m downstream of the water turbine. The tank will be of port type. Its dimension is shown in Plate P17, and the surging wave curves are shown in Fig. 8.4.

8.6 Power House

The power house will be of underground type. It will be constructed inside a limestone block, located across the Erik river near the confluence with the Ermenek river. The layout of access tunnels is shown in Plate P18. A layout plan of the power house is shown in Plate P19.

The turbine center will be EL. 320 m, being 13 m lower than the normal tailwater level. The lowest elevation below the draft tube will be EL. 310.5 m, which will be 6.5 m above the bottom surface of the limestone block.

A cavern of 38.5 m high, 27.0 m wide and 98.0 m long will be constructed by NATM without concrete lining. The draft tube gate chamber will be constructed between the power house and the tailrace chamber.

8.7 Erik Diversion Scheme

The Erik Diversion Scheme will consist of an intake weir, a desilting basin, a diversion tunnel, a head tank with an excess water spillway, a penstock, a power house, an inlet shaft with an air-trap chamber, and a connecting tunnel (see Plate P20).

(1) Intake weir

The Erik intake weir will be located in a limestone block (Plate P21). There is a large scale active landslide on the left bank about 100 m downstream of the block. It is expected that this land would move during heavy rainfall to block the Erik river. In such a case, debris flow will rush down the river after the natural dike is breached by the accumulated flow. To protect the weir from such an attack of debris flow, the weir site was selected upstream of the landslide area.

The Erik intake will be of Tirolean type, which will trap the flowing water over trashracks (see Plate P21). The design discharge of intake is 6.0 m^3/s. The intake weir will be of concrete gravity type. The design flood is 400 m^3/s as 100-yr probable. The crest elevation of the weir will be set at 820 m, being about 3 m higher than the elevation of Erik spring. This crest elevation was provisionally selected in order to have a hydraulic head necessary to flush sand deposited in the desilting basin. The crest bridge will be submerged during the weir design flood.

(2) Desilting basin

In the site reconnaissance made in July 1989, cobble stones were observed on the riverbed, which was the outcrop of limestone block. The Erik flow was quite cool being about 11^{o}C and was clear. However, there is no data on the sediment transport of the upper Erik river during flood and snow-melting periods.

A desilting basin is planned to be provided assuming that there are suspended sediment during floods and snow-melting season. Because of the site topography, the basin will be located underground (Plate P21). The Erik flow trapped at the intake weir will be guided to the desilting basin through a connecting tunnel, where an intake gate and a side-overflow weir for excess water will be provided. The sediments deposited on the basin bottom will be flushed through a sand flush tunnel by manual operation of a sand flush gate provided at the downstream end of the basin. The sand flush spillway will be used also as an excess water spillway. After desilted, the water will overflow a crest provided near the downstream end of the basin and will enter the Erik diversion tunnel.

To prevent the electromechanical facilities from water submergence and excessive inflow into the diversion tunnel in case of the above-mentioned landslide, a tailrace gate will be provided in the sand flush tunnel. This and intake gates will be remotely-supervised and operated from the Ermenek power station.

(3) Erik diversion tunnel

The Erik diversion tunnel will be of free flow type. It will have a semi-horse shoe section with a width of 2.2 m and a height of 2.3 m. The length will be 3,580 m. The tunnel route was selected detouring the landslide (Plate P22). The water depth in the tunnel will be about 1.6 m at the design discharge of 6 m^3/s for a hydraulic gradient of 1/1,000 and a coefficient of roughness of $n = 0.014$. The design clearance between the water surface and tunnel crown will be 0.7 m.

(4) <u>Headtank and excess water spillway</u>

A headtank will be provided to have a free water surface before the flow enters the penstock. The surface area was determined to be 60 m^2 as 10 times the design discharge; the effective capacity to be 120 m^3 as 20 times the design discharge. The normal HWL will be 812 m and LWL will be 810 m with a drawdown of 2 m (Plate P23).

An excess water spillway will be provided in parallel to the penstock (Plate P24). This will guide the flow to the tailrace chamber when the Erik power station is not in operation. The excess water will be spilt from a side overflow weir. The weir will have a crest elevation at 812 m, a crest length of 12 m, a design head of 0.4 m, a design discharge of 6 m^3/s. The maximum water level of the headtank will be 812.4 m during the full discharge from this overflow weir. Against this water level, the diversion tunnel will still have a clearance of 0.3 m between the water surface and the tunnel crown.

(5) <u>Erik penstock</u>

The penstock will be of surface type; 240 m in length; 1.2 m in internal diameter (Plate P24).

(6) <u>Erik power station</u>

The power house will be of surface type (Plate P25). The power house will be 18.5 m wide, 18.2 m long, 24.5 m high. It will accommodate one set of generation equipment. The normal tailwater level will be 675 m, being equal to the HWL of the Ermenek reservoir. The turbine center elevation will be 674.40 m.

During surging in the Ermenek headrace tunnel, the water level in the Erik tailrace chamber will be affected. The maximum water level will be 693.93 m in the full load shut-down; the minimum water level will be 592.08 m in a half load increase condition. The top elevation of the tailrace chamber will be set at 695.5 m with a freeboard of 1.57 m. The tailrace chamber will be 25 m in height above foundation; 12.5 m in height above the ground surface; 8 m in internal diameter.

(7) Inlet shaft

The inlet shaft will be inclined at 45 degrees. The inclined portion will be 90.43 m in height. The section will be semi-horseshoe with a width of 3.5 m and a height of 3.5 m. The invert elevation will be 578.62 m at the bottom of the shaft.

An air trap chamber will be provided in the horizontal tunnel connected to the Ermenek headrace tunnel. Immediate upstream of the joining point to the headrace tunnel, a throttle device will be provided to reduce the reverse flow into the shaft during surging.

8.8 Hydromechanical Works

The principal items of hydromechanical works for the Project will consist of gates, stoplogs, trashracks, steel liners and penstock. The locations of the proposed hydromechanical works are shown in the plates for respective structures.

(1) Gates, stoplogs and trashracks

The type of various gates was selected based on the respective dimensions, water pressures acting on the gate, functional requirements and costs. Fixed wheel gates and slide gates will be adopted as described in item (3) below.

(2) Steel liner for the pressure shaft

Based on the design of pressure shaft, the thickness of steel shell plate was designed depending on the acting water pressures including those due to water hammer.

In the design, the following classes of steel materials were considered;

Steel Class	Allowable Stress (kgf/cm^2)
JIS SM41	1,300
JIS SM50	1,750
JIS SM58	2,400

The thickness of steel shell plate was calculated to be 18 mm to 38 mm depending on the acting pressure. The total weight of the steel structures is estimated to be about 3,600 tonnes.

(3) Principal features of hydromechanical works

(A) Dam and intake facilities

(a) Diversion gate

Type:	Slide gate
Quantity:	1 set
Dimensions:	7.0 m wide x 7.0 m high
Design head:	50 m

(b) Tunnel spillway gates

Type:	High pressure fixed wheel gate
Quantity:	2 sets

Dimensions:	3.0 m wide x 7.0 m high	
Design head:	45 m	
Appurtenant equipment:	Stoplog	1 set
	Steel liner	1 set

(c) Power intake gate

Type:	High pressure fixed wheel gate	
Quantity:	1 set	
Dimensions:	6.1 m wide x 6.1 m high	
Design head:	75 m	
Appurtenant equipment:	Fixed trashrack	1 set
	Stoplog	1 set

Note: Raking equipment will not be provided.

(d) Bottom outlet facility

<u>Main gate</u>

Type:	High pressure slide gate
Quantity:	2 sets
Dimensions:	2.5 m wide x 4.0 m high
Design head:	130 m

<u>Guard Gate</u>

Type:	High pressure slide gates	
Quantity:	2 sets	
Dimensions:	2.5 m wide x 4.0 m high	
Design head:	130 m	
Appurtenant equipment:	Steel liner	2 sets

(B) Steel liner for pressure shafts

Type:	Embedded steel liner
Quantity:	2 lanes
Dimensions:	3.6 m in diameter

Design head: 415 m at turbine center

(C) Power house facilities

(a) Draft tube gates

Type:	Slide gates operated by hydraulic hoist
Quantity:	2 sets
Dimensions:	5.5 m wide x 5.0 m high
Design head:	30 m

(b) Tailrace gate

Type:	Slide gates
Quantity:	1 set
Dimensions:	6.1 m wide x 6.1 m high
Design head:	8 m

(D) Erik Diversion Scheme

(a) Intake weir trashrack

Type:	Fixed trashrack
Quantity:	1 set
Dimensions:	7.0 m wide x 4.0 m long
Design head:	4.5 m

(b) Intake gate

Type:	Fixed wheel gate
Quantity:	1 set
Dimensions:	2.2 m wide x 2.3 m high
Design head:	30 m

(c) Sand flush gate

Type:	Fixed wheel gate, manual operation
Quantity:	1 set
Dimensions:	2.0 m wide x 2.0 m high
Design head:	5 m

(d) Sand flush tunnel closing gate

Type:	Fixed wheel gate
Quantity:	1 set
Dimensions:	2.0 m wide x 2.0 m high
Design head:	35 m

(e) Headtank trashrack

Type:	Fixed trashrack
Quantity:	1 set
Dimensions:	4.0 m wide x 5.4 m high
Design head:	6 m

(f) Headtank drain gate

Type:	Fixed wheel gate
Quantity:	1 set
Dimensions:	1.0 m wide x 1.0 m high
Design head:	6 m

(g) Steel penstock

Type:	Exposed steel penstock
Quantity:	1 lane
Dimensions:	1.2 m in diameter
Design head:	151 m at turbine center

8.9 Generation Equipment

(1) Power station arrangement

Two sets each of hydraulic turbine, generator and step-up transformer with their auxiliary equipment will be installed in the underground powerhouse. A 380 kV switchgear will be arranged in the outdoor switchyard. 380 kV power cables will be installed along the access tunnel to connect the underground powerhouse and the outdoor switchyard. A single line connection diagram of the power station is shown in Plate P27. A general plan of underground powerhouse arrangement is shown in

Plate P19 and a layout of the outdoor switchyard in Plate P28.

Two sets of overhead travelling crane of 135 ton capacity will be provided in the powerhouse for handling heavy power station equipment such as hydraulic turbines, generators, main transformers and so forth.

(2) Number of generating units

The number of generating units for the total installation of 320 MW with the rated hydraulic head of 308 m is selected to be two from considerations below:

(A) The total construction cost will decrease by reducing the number of generating units to be installed. The total cost will be cheapest for the one unit installation.

(B) From the operation aspects, the number of equipment had better be two or more. The minimum operating output of Francis turbine is normally about 40 per cent of the rated output, and accordingly the minimum operating output (320 MW x 0.4 = 128 MW) under one unit installation will be too large to adjust the operating outputs in accordance with the system requirement especially during the off-peak time. In addition, one unit outage under the one unit installation will result in the total outage of the station.

(C) The total capacity of the Turkish power system and that of the neigbouring region is large enough and it will not require an installation of 3 units or more.

(3) Hydraulic turbines

Considering the working head and rated output, the hydraulic turbines will be of vertical shaft Francis type and their particulars will be as mentioned below:

(A) Hydraulic conditions

- Reservoir water level	
Maximum	: 675 m
Minimum	: 615 m
Rated (average WL, 65 % in probability of exceedence)	: 660 m
- Tail water level	: 333 m
- Gross head	
Maximum	: 342 m
Minimum	: 282 m
Rated	: 327 m
- Rated net head	: 308 m
- Maximum discharge	: 116.6 m^3/s

(B) Hydraulic turbines

- Type	: Vertical shaft, Francis
- Rated head	: 308 m
- Number of units	: 2
- Rated output	: 163.5 MW
- Rotation speed	: 333 rpm

The hydraulic turbines will be designed so that the machines can be operated at an output 5 per cent larger than the rated one under an operating head higher than the rated one.

(4) Generators

The generator will be vertical shaft alternators to be directly coupled with the hydraulic turbine with particulars as given below:

- Type	:	Vertical shaft, revolving field, 3-phase, synchronous alternator
- Number of units	:	2
- Excitation system	:	Static thyrister type
- Rated output	:	180 MVA
- Rated voltage	:	14.4 kV
- Rated power factor	:	88.9 per cent under normal operation 93.3 per cent under 5 per cent overload operation
- Rotation speed	:	333 rpm

(5) <u>Main transformers</u>

For the main transformers to step up the generator output voltage to the 380 kV transmission voltage, single phase transformers will be adopted in order to limit transportation weight. Three single phase transformers will be installed close to each other and will form one bank. Particulars of the transformers will be as follows:

- Type	:	Single phase, two winding
- Cooling system	:	Water cooled, forced oil circulation
- Capacity	:	60 MVA
- Number of units	:	6 (3 units x 2 banks)
- Voltage ratio	:	14.4/380 kV
- Tap changer	:	Off-load, plus and minus 10 per cent

(6) Power cables

380 kV cables of CV (cross-linked polyethylene insulated and PVC sheathed) type or oil-filled type will be installed along the access tunnel, to connect the main transformers in the underground powerhouse with the outdoor switchyard.

(7) Outdoor switchyard

A 380 kV switchgear for connection with transmission lines will be arranged in the outdoor switchyard to be located near the entrance of the access tunnel. The bus system of the switchyard will be of double bus with a transfer bus.

(8) Control system

A one-man-control system will be employed for controlling the generation equipment. Both the generation equipment in the underground powerhouse and a 380 kV switchgear in the outdoor switchyard will be controlled from the control room to be located in the underground powerhouse on the elevation higher than the machine room so that the generation equipment can be seen from the control room.

Another control house will also be provided at the outdoor switchyard to control the 380 kV switchgear. Relaying equipment and local control equipment will be installed in the control house. However, under normal operation, no operators will be stationed in the outdoor control house as the 380 kV switchgear will be remotely-supervised and controlled from the underground control room. Data communication between the underground control room and the outdoor control house will be made through optic fibre cables.

8.10 Transmission Line

As mentioned in Sub-section 3.5.1, the transmission line to be constructed under the Project will be a 380 kV transmission line between the Ermenek power station and the Seydişehir substation. On the Seydişehir side, the 380 kV line will be terminated at the existing substation bus.

This transmission line will be provided with 3-bundled 954 MCM ACSR, which are the standard conductor size of main 380 kV lines in Turkey. The current carrying capacity of this line is about 2,600 A and the allowable power capacity is about 1,700 MVA.

Except for the limited sections approaching the Seydişehir substation, the proposed line route will run in mountainous area along national roads. However, the mountains are generally not so steep and there will not be much difficulty in approaching most of the line route by motor vehicles. The proposed route of the transmission line is shown in Plates P31 and P32.

The line protection system will be provided with two fully independent distance protection schemes with different types of static relays. Main 1 protection will be directional comparison blocking scheme and Main 2 protection will be permissive underreach transfer tripping scheme.

The voice and data communication on the transmission line will be provided with the power line carrier (PLC) system.

8.11 Generation Equipment of Erik Power Station

(1) Overall arrangement

Generation equipment of the Erik power station will be arranged in the surface type powerhouse, which will be provided with a small overhead travelling crane. A step-up transformer and a 34.5 kV switchgear will be placed outdoors adjacent to the powerhouse. The proposed arrangement is shown in Plate P25.

(2) Number of hydraulic turbine-generator

Against the maximum discharge of 6 m^3/s, the 90% dependable discharge is 2.85 m^3/s (47.5 %) and the minimum discharge is 2.14 m^3/s (35.7 %). This variation in discharge can be covered by one unit Francis turbine. For the rated output of 6,700 kW, total construction cost of one unit installation will be much cheaper than that of two unit installation. Therefore, one unit installation will be selected for this power station.

(3) Hydraulic turbine

One set of vertical shaft Francis turbine will be installed in the powerhouse.

(A) Hydraulic conditions

- Intake water level : 820 m
- Tail water level : 675 m
- Gross head : 145 m
- Rated head : 133 m
- Maximum discharge : 6 m^3/s

(B) Turbine details

- Type : Vertical Francis
- Output : 6,950 kW

- Rotation speed : 750 rpm

The intake water level was initially assumed at EL. 815 m, being about 2 m below the elevation of Erik spring. This water level was later raised by 5 m to EL. 820 m in order to have a hydraulic head necessary to flush sediments deposited in the desilting basin. As a result, there is a possibility to increase the above rated head of 133 m to about 138 m.

The horizontal type of turbine-generator is also conceivable. However, the vertical type was tentatively selected as its required floor area is smaller than that for the horizontal one.

These matters need thorough review and examination at the final design stage.

(4) Generator

The generator will be directly coupled with the hydraulic turbine and its particulars will be as given below:

- Type	:	Vertical shaft, revolving field, 3-phase, synchronous alternater
- Excitation system	:	Static thyrister
- Number of phase	:	3
- Output	:	8,375 kVA
- Rated power factor	:	0.8
- Terminal voltage	:	6.6 kV
- Rotation speed	:	750 rpm

(5) Power station control

As the Erik power station will be operated automatically depending on the available flow of the Erik river, no daily control operation will be required under normal operation. Therefore, it is planned to operate this power station with remote supervision and control from the control room of the Ermenek power station. Data and signal for supervision and control will be transmitted through a telephone cable.

However, local control equipment will also be provided for testing and maintenance operations.

(6) <u>Other power facilities</u>

One set of 6.6/34.5 kV, 8,375 kVA transformer and one lot of 34.5 kV switchgear will be installed outdoors adjacent to the powerhouse for stepping up the generator voltage to the 34.5 kV line voltage and for taking out one 34.5 kV power line. The 34.5 kV line from the Erik power station will be connected to the 34.5 kV line which will be constructed between the Ermenek power station and the Ermenek dam/Erik intake weir sites. This line will further be connected to the 34.5 kV line existing between Ermenek and Kazançi for supply to the public demand. Plate P27 is referred to regarding the connection of the Erik power station with the main power station.