m, but at deeper than 10 m low permeabilities of Lu from 0 to 3 were generally indicated. On the other hand, comparatively high values of Lu = 3 to 10 were indicated to a depth of 30 m at the top.

- Diversion Tunnel

In results of Lugeon tests at Drillholes TSK-1, -2, -3, and -4, low permeabilities of Lu = 0 to 3 were indicated at all sections along the projected elevation of the diversion tunnel of approximately 340 m.

iv) Laboratory Tests

DSİ dis unconfined compression tests for 3 core samples of SK-2 and 2 core samples of TSK-3 in order to get the fundamental data to assess the bed rock strength at the Dam Site.

The test results are shown in Table 7-8.

Table 7-8 Unconfined Compression Test at Dam Site

Drill hole No.	Depth (m)	Strength (kgf/cm^2)	Dimension (cm)
SK-2	74.00 ~ 74.30	335	ϕ=6.2 h=12.4
SK-2	74.30 ~ 74.60	160	ϕ=6.2 h=12.4
SK-2	93.20 ~ 93.50	290	ϕ=6.2 h=12.4
TSK-3	90.30 ~ 90.80	330	ϕ=6.2 h=9.6
TSK-3	90.30 ~ 90.80	800	ϕ=6.2 h=12.4

v) Rock Classifications

The following method was adopted for rock classifications used in the engineering assessments which also included the waterway and the powerhouse.

Regarding boring cores, as shown in Table 7-8, the rock cores were classified according to 5 stages regarding weathering degree (W), hardness (H), and crack interval (C), and these are recorded in boring logs (see Appendix A-3-1). Furthermore, these classification results, as shown in Table 7-7, give core evaluations according to groupings, and these are illustrated in geological profiles and geological logs in Figure 7-6 to 16. This core evaluation was used here for bedrock classification.

vi) Engineering Geology

1. The granitic rocks distributed at the dam site have cracks developed at their surface portions, but if strongly weathered parts (rock evaluation ⓓ and ⓔ are removed, it is judged there will be ample bearing capacity as a foundation for a fill dam of 100-m class. These strongly weathered surface layer portions are 10 m from the surface at the left-bank abutment, approximately 20 m at mid-height of the left bank, 30 m from the surface at the right-bank abutment, and approximately 10 m at mid-height of the right bank, to be slightly thick at the right-bank abutment. Rather deep weathering at the right-bank is considered to be due to strongly altered granodiorite distributed near the ground surface.

   At the spillway, cracks are developed at approximately 12 m from the ground surface at the top and approximately 7 m from the ground surface at the downstream slope, and parts of rock evaluations ⓓ and ⓔ are distributed, but these are not as deep as at the right-bank slope of the dam. The bedrock at the elevation at which the diversion tunnel passes is of bedrock evaluation ⓑ and ⓒ for a good condition.

2. As discontinuous planes in the foundation rock there are faults, joints, and intrusion planes of diabase. As previously mentioned, the faults confirmed at the dam site and the spillway section are of small scale, do not have so long continuity, and are not accompanied by prominent fault gouge or fault breccia. Therefore, prominent faults which demand special treatment are not ascertained.

   Joints are comparatively developed, and weathering discoloration along joint planes is seen to around depths of 20 to 30 m from the ground surface at the left-bank side and around 10 to 60 m from the ground surface at the right-bank side.

   The two joint systems of N50° W50° NE, and N10° E45° E seen at the left-bank abutment are tending to dip along the slope, and since small collapses are seen at the slopes along the existing road due to these joint systems, care will need to be paid when excavating for the dam slope.

   The intrusion planes of diabase are completely tight, zones with cracking developed along the planes are not present, and gouge and breccia are not seen.

   It is thought that the hair cracks developed along schistose structures of altered zones are considered not to be of much problem for the foundation of a fill-type dam from the point of view of deformability and permeability.

3. According to the results of Lugeon tests, slightly high permeability of Lu = 3~10 down to depths of 20 to 30 m from the ground surface is indicated at the river-bed portion and right bank of the dam, but deeper than this, except at an extremely small part, permeability is low at Lu = 0~3. At the spillway, the permeability is slightly high at Lu = 3~10 to the depth of about 30 m. At the elevation at which the diversion tunnel passes, permeability is low with Lu = 0~3 at all points.

   The bedrock of this site consists of hard granitic rocks, there is no water seeping through between the mineral grains of the rocks, and permeability, according to discoloration at fissures and results of Lugeon tests, is considered to be governed by the previously-mentioned discontinuities of joints. Further, since there are no high-permeability zones at deep parts, it is thought possible for adequate cutoff treatment to be provided by excavating and removing parts near the ground surface where cracks are developed and are strongly weathered (rock evaluation d and e, and carrying out cement grouting of the kind generally employed from the ground surface.

4. The river-bed portion of the foundation for the rock zone of the dam is covered with river-bed deposits consisting of gravel and coarse-grained sand to a thickness of approximately 10 to 20 m, while the slopes at the left and right banks have topsoil approximately 5 m in thickness and

contain gravel in parts. It is necessary for these parts to be excavated and removed. Topsoil several meters in thickness is distributed at the spillway section also.

5. To summarize the above results, it appears there are no engineering geological conditions which will pose major problems hereafter for structures such as the dam, spillway, and diversion tunnel.

b) Alternative Dam Site

i) Topography

A site approximately 1 km upstream from the dam site of the Master Plan is conceivable as an alternative dam site. The valley width of this dam site is narrow compared with the Master Plan site with river-bed width approximately 40 m and crest width approximately 360 m (at EL. 437 m). Both banks are slopes inclined approximately 30° for a shape comparatively close to a V. The right-bank side is a slender, scraggy ridge (saddle at EL. 435 m).

ii) Geology

There is a comparatively large number of outcrops of the basement rock at the alternative dam site and its vicinity, the basement rock of this site consisting of granitic rocks of the Mesozoic Jurassic Period and diabase intruded in it at parts, similarly to the Master Plan dam site. Both rocks, so far as seen at outcrops, are slightly weathered, but compared with the Master Plan dam site, the degree of weathering is less due in part to the slightly rugged topography, and as a whole, it appears rock of comparatively fresh character can be expected at a depth several meters from the ground surface.

Surface deposits are thin on the whole at the slopes of both banks of the alternative dam site and the ridge on the right bank, and the thickness is thought to be not more than 1 m. River-bed sand-gravel is distributed along the flow channel at the river bed. As seen from the topographical features of the flow channel and the river bed, it is thought the distribution condition of river-bed deposits is about the same or of smaller scale in comparison with the Master Plan dam site.

iii) Engineering Geology

There appears to be no fatality problem in particular from an engineering geological standpoint, but there is a scraggly saddle part way along the ridge at the right-bank side, and depending on how the high water level is set it will be necessary to construct an auxiliary dam at that location. In construction of the auxiliary dam, since the foundation will be located at the scraggly ridge, a careful investigation concerning mechanical stability and permeability of the basement rock will be absolutely necessary.

(3) Waterway and Powerhouse

For the waterway and powerhouse sites, the alternatives of D Layout (Underground Powerhouse - Tailrace Tunnel Type) and A Layout (Headrace Tunnel - Open Air Powerhouse Type) as shown in Figures 7-1, -12, -13, and -14 were examined and D Layout was selected in the end from the standpoint of the economics.

a) D Layout (Underground Powerhouse - Tailrace Tunnel Type)

i) Topography

The intake would be provided at the EL 385 m point at midheight of a slope inclined approximately 30° on the south side of the right-bank ridge immediately upstream of the dam, from where the headrace will extend horizontally 65 m in the direction of the core of the ridge (to the north). Then, parallel to the north slope of the right-bank ridge, a penstock of approximately 265 m will run downward at an inclination of 45° in the direction to the north with rock cover of approximately 200 m to reach the projected underground powerhouse site. The projected underground powerhouse site is deep inside the mountain body at the end of the right-bank ridge's north slope (with rock cover of the arch section of approximately 170 m).

From there, a tailrace of tunnel-type for approximately 4900 m and an open canal-type for approximately 200 m continues in the northwest direction at an elevation of approximately 225 m. The route of the tailrace tunnel consists on the whole of a gently sloped mountainland, the section of approximately 4000 m on the powerhouse side having cover of approximately 100 to 300 m, with the section of approximately 1000 m on the outlet side having cover of not more than approximately 100 m, which is about 20% of the whole tunnel length. The open canal-type tailrace of approximately 200 m is located on the flood plain of the Devrek River at an elevation of approximately 225 m. There are a dozen or more gullies crossing over the tailrace tunnel, but all are small, and the rock cover at the gullies is over 150 m high above the tunnel.

ii) Geology

The route of intake, headrace, penstock, underground powerhouse, and tailrace tunnel consists of granitic rocks of the Mesozoic Jurassic Period intruded at parts by diabase as shown in Figure 7-12, 13, 14.

Fresh bedrock is hard and dense. The granitic rock, according to microscopic observations of thin sections, is an altered granodiorite having a holocrystalline structure, consists mainly of plagioclase, quartz, hornblende, and biotite, and having been subjected to weak crushing and altering actions, it is accompanied by chlorite, sericite, and chlorite carbonate minerals. Compared with the altered granodiorite distributed at the dam site, it is generally coarse-grained, and except for parts which have been subjected to hydrothermal alteration, the degree of alteration is small.

At the open canal-type part of the tailrace there are distributions of river deposits of the Quaternary Period, and deeper under them, talus deposits, with no bedrock down to a depth of 25 m at Drillhole DD-4.

Hydrothermal alteration is seen prominently at the ground surface for approximately 800 m at the middle part of the headrace tunnel and for approximately 800 m in the vicinity of the outlet, and rock bodies at these parts are extremely deteriorated. In X-ray diffraction tests of these altered parts, there have been large quantities of calcite, minute quantities of montmorillonite, and extremely minute quantities of kaolinic minerals detected, and it can be estimated that hydrothermal alteration had occurred. It is thought that this hydrothermal alteration is related to action of the igneous rock (porphyrite) distributed at the west side of the tailrace.

The only prominent fault is the one in the vicinity of Drillhole DD-2 roughly orthogonal to the tailrace tunnel having a strike in the north east-southwest direction and a dip of 85° to the northwest, but in aerial photographs there are discerned 3 lineaments having strikes in the north-south direction and intersecting diagonally with the tailrace tunnel. Prominent fracture zones found in boreholes drilled for D Layout are those two at depths of 60 to 65 m in DD-2 and 22 to 25 m in DD-3.

Joints are comparatively well-developed, with predominant directions being the three of EW70°N, N30°E70°NW, and N20° W60°SW. Fissure spacings in boring cores, except for fault fracture zones and surroundings, are around 3 to 10 cm at waterway passage routes, and from 10 to 30 cm at the underground powerhouse site. RQD values except for around fracture zones, are about 50% (Fair~Poor) at waterway passage routes, and roughly 100% (Excellent) at the underground powerhouse site.

Weathering from the ground surface is shallower than 30 m except for around fault fracture zones.

Large-scale landslides are not seen to have occurred in the area where structures are to be distributed.

iii) Hydro geology

1. Groundwater Level

The final water levels in holes drilled in the vicinities of the waterway and powerhouse sites are shown in Figures 7-15 and -16 and the variations in groundwater levels during drilling in Figure 7-14.

The water level in Drillhole DDV-1 (L = 270 m) made in the vicinity of the projected underground powerhouse site was more or less constant at a depth of approximately 40 m during drilling, while the final water level in the hole was at a depth of 59 m. Drillhole DD-2 (L = 80 m) had a water level close to the ground surface during drilling and the final water level in the hole was at a depth of 0.8 m. Drillhole DD-3 (L = 50 m) had a final water level at a depth of 12 m, corresponding to the bottom part of a strongly weathered zone. Drillhole DD-4 (L = 25 m) had a final water level at a depth of 2 m, roughly equal to the water level elevation of the Devrek River.

2. Permeability

Lugeon tests using drillholes were carried out at the headrace and dam sites on a total of 5 holes, 82 stages, and a length of 165 m as shown in Figures 7-15 and -16 and Appendix A-3-3. These tests were conducted at test stages of 2 m intervals to estimate the permeabilities and water-springing conditions of foundation rock at the planned depths of the projected waterway tunnel and powerhouse sites.

The methods of the Lugeon tests were of general kinds, but since the projected underground powerhouse site is to be at a depth of more than 200 m, testing was done raising the maximum pressure to 20 kgf/cm^2. The results were analyzed by the same method as for the dam site.

The results of Lugeon tests on Drillhole DDV-1 provided in the vicinity of the projected underground powerhouse site were that inflow was practically zero even when maximum pressure was raised to 20 kgf/cm^2, except around the depth of 200 m where cracks were developed. The part of Drillhole DD-2 lower than a depth of 50 m indicated low permeability of around 1 Lu.

iv) Laboratory Tests

DSI did Unconfined compression tests for 3 core samples of DDV-1 in order to the fundamental data to assess the bed rock strength at the underground power house. The test results are shown in Table 7-9.

Table 7-9 Unconfined Compression Test at Underground Power House

Drill Hole No.	Depth (m)	Strength (kgf/cm^2)	Dimension (cm)
DDV-1	236.00 ~ 236.18	899	ϕ=4.6 h=8.2
DDV-1	236.20 ~ 236.40	869	ϕ=4.6 h=9.2
DDV-1	266.50 ~ 266.62	395	ϕ=4.7 h=9.0

v) Geophysical Prospecting

DSI did the geophysical prospecting (seismic refraction method) at the A-Layout route and the D-Layout in order to assess the bed rock engineering geological characteristics. The location of the rootes and the prospecting results are shown in Appendix A-3-5. And the characteristics of the typical classes of rock mass are summarized in Table 7-10.

Table 7-10 Relation between Geological Layers and Vp

Symbol	type	P Wave Velocity (m/sec)
1a	Clayed soil, talus	375 - 630
1b	Alluvion(gravel, sand, clay, silt)	500 - 1914
2a(e)*	Highly weathered granodiorite	610 - 1115
2b(d)*	Weathered granodiorite	1021 - 1893
2c(d)*	Highly Fissured granodiorite	1850 - 2684
2d(c)*	Fissured granodiorite	3112 - 3589

* Rock classification

The result of the survey at the D-Layout route shows that there are the 1a layer, 10~15 m thickness below the surface, the 2b and 2c layers, about 20 m thickness, below the 1a layer, and the 2d layer below them. The layers distribute parallel to the topographical surface.

vi) Engineering Geology

1. The intake is planned at a granodiorite outcrop on a slope of approximately 30° at midheight of the right-bank ridge's south slope immediately upstream of the dam. Topographically, landslides, collapses, etc. are not seen and it is a stable slope. The granodiorite distributed there is comparatively coarse-grained and the degree of alteration is low. The rock is fresh, hard, and dense even at outcrops. As a result of aerophoto interpretations, a lineament was discerned extending northwest-southeast in the vicinity of the intake, but a large-scale fault corresponding to this was not confirmed in field reconnaissances. Joints with strikes in the north-south direction and dips close to vertical are predominant.

2. The headrace and penstock are planned to cross the ridge on the right-bank side in relation to the dam, and the rock cover is more than 200 m except for extremely small parts of the headrace and penstock. The tunnel route is a distribution zone of comparatively coarse-grained granodiorite of small degree of alteration and the rock quality is generally fresh, hard, and dense, and except for fracture zones at faults, good bedrock of Class C or better can be expected. Prominent faults intersecting the tunnel route are not ascertained.

3. The projected underground powerhouse site is located deep inside the mountain body (rock cover of the arch section approximately 170 m) on the north slope at the end of the ridge on the right-bank side in relation to the dam, and is a distribution zone of granodiorite of a low degree of alteration, with the rock quality generally being fresh, hard, and dense, and a good bedrock of Class C or better can be expected. Fissure spacings are from 10 to 30 cm in boring cores, and RQD is roughly 100% (Excellent). Hardly any weathering can be seen. Prominent faults have not been confirmed in Drillhole DDV-1 provided at the projected underground powerhouse site. At the projected depth of the underground powerhouse, water levels in drillholes being drilled were stable in the neighborhood of a depth of 60 m, and there were no pervious strata. In permeability tests there was practically no inflow, with Lugeon value being zero in most cases. It is thought from the above that there will be no great springing of water during excavation.

The average value of the unconfined compression tests is 721 kgf/cm^2. This value is said to be enough one to construct an underground power station cavern. The values vary widely; therefore, at the definite design stage DSÎ had better do the additional tests to assess the more detailed stability of the cavern.

4. The tailrace tunnel route consists as a whole of a mountainland of gentle slope and is a granodiorite distribution zone. Rock cover is generally from approximately 100 to 300 m, with the section of approximately 1000 m on the outlet side having a rock cover of approximately 100 m or under. Except for those sections of thin cover, fracture zones at faults and hydrothermally altered parts, rock quality is generally hard and dense, and a bedrock of about Class C can be expected.

There is a comparatively large number of cracks with spacings 3 to 10 cm in boring cores, RQD values being around 50% (Fair~Poor). The only prominent fault intersecting the tailrace tunnel is the one roughly orthogonal to the tunnel in the vicinity of Drillhole DD-2 having a strike of northeast-southwest and dip of 85° to the northwest, but in aerial photographs there are 3 lineaments of north-south strike intersecting the tunnel diagonally and there is a possibility that these comprise fractured zones in the bedrock.

Hydrothermal alteration is prominently seen at the ground surface for a section of approximately 800 m at the middle part of the tailrace tunnel route and a section of approximately 800 m at the vicinity of the outlet, and it is thought this hydrothermal alteration has caused the bedrock at the elevation of the tunnel route to be extremely deteriorated.

To summarize the above results, there is deterioration of bedrock to the extent of hindering tunnel excavation at sections of thin rock cover, the fracture zone of a fault, weathered zone, and hydrothermal alteration, and the length of these together is thought to be one third of the entire length.

According to the results of permeability tests, permeability is low at 3 Lugeons and under, and except for around the fault, it is thought there will be no large-scale water springing at depths of thick cover.

5. The tailrace of open-canal type is located on the flood plain of the Devrek River, and there is distribution of Quaternary river-bed deposits at the ground surface with talus deposits distributed at deep parts and there is no anchorage on rock down to a depth of 25 m. When the surface soil is removed, the river-bed deposits has the enough strength as the foundation.

b) A Layout (Headrace Tunnel - Open Air Powerhouse Type)

i) Topography

The intake is to be provided at a site at EL 385 m on a slope inclined approximately 30° at midheight of the south slope of the right-bank ridge immediately upstream of the dam similarly to D Layout, from where the headrace tunnel of total length approximately 5500 m will run roughly horizontally at an elevation of about 360 m, first going approximately 100 m toward the core of the ridge (northern direction) and then changing direction to the northwest. The route of the headrace tunnel is in gently sloped mountainland as a whole, the rock cover being approximately 60 to 160 m and thin compared with D Layout. There are a dozen or more gullies crossing the headrace tunnel, and all are small, but since rock cover is thin from the beginning, the cover thickness around gullies will be less than about 100 m. When the low-elevation part around the surge tank is included the cover for approximately half of the total length of the headrace tunnel (L = 2,700 m) will be less than about 100 m.

The surge tank is planned in the vicinity of EL 450 m of a ridge extending north-south and gently sloping to the north. From the surge tank there will continue a penstock which extends north toward the core of the ridge's mountain body, for approximately 400 m as a tunnel type and then for approximately 500 m as an open-air type. The rock cover of the tunnel-type penstock will be less than about 80 m. The inclination of the ridge where the open-air type penstock is planned to run is approximately 15 to 20°.

The powerhouse is planned as an open-air type at the end of this ridge. And the open canal-type tailrace of approximately 480 m would be located on the flood plain of the Devrek River at an elevation of approximately 225 m similarly to D Layout.

ii) Geology

Similarly to D Layout, the route of the intake, headrace tunnel, surge tank, penstock, open-air type powerhouse, and tailrace, as shown in Figures 7-12 and -13, consists of granitic rock of the Mesozoic Jurassic Period intruded at parts by diabase. Fresh part of the bedrock is hard and dense. This granitic rock has the same rock character as in D Layout.

The open canal-type tailrace has Quaternary river-bed deposits at the ground surface, with weathered granitic rock distributed deep underneath.

Hydrothermal alteration is seen at the ground surfaces of an approximately 800 m section at the middle part of the headrace tunnel route, an approximately 800 m section in the vicinity of the surge tank, and the entire area of the penstock route, and the rock bodies of those parts are greatly deteriorated.

The only prominent fault is the one of a northeast-southwest strike, dipping 85° to the northwest intersecting the headrace tunnel diagonally approximately 800 m toward the dam side from the surge tank, but in aerial photographs there are discerned 3 lineaments having strikes in the north-south direction and crossing diagonally with the headrace tunnel. Prominent fracture zones found in drillholes drilled for A Layout are those four at depths of 52 to 53 m and 65 to 66 m in DA-2, and at depths of 22 to 25 m and 36 to 39 m in DA-3.

Joints are comparatively well-developed, with predominant directions being the three of EW70°N, N30°E70°NW, and N20°W60°SW. Fissures are very numerous in the boring cores from elevations passed by the tunnel, the spacings being about 1 to 10 cm, with RQD values 20% and under (Very Poor).

Weathering from the ground surface is generally shallower than 30 m except for around fault fracture zones, and at the DA-2 (L = 70 m) and DA-3 (L = 70 m) holes located in hydrothermal alteration zones weathering has extended close to the hole bottoms.

Large-scale landslides are not seen to have occurred in the area where structures are to be distributed.

iii) Hydro geology

The water level in Drillhole DA-1 (L = 50 m) while drilling was more or less constant at approximately 8 m, and the final water level in the hole was at approximately 6.5 m, the boundary portion between talus deposits and the basement rock. The water level difference before and after drilling of DA-2 (L = 70 m) was large with approximately 10 m at 50 m and shallower, and the final water level in the hole was at approximately 25 m. At Drillhole DA-3 (L = 70 m), the water level difference before and after drilling was large at practically all of the sections at over approximately 10 m, and the final water level in the hole was deep at approximately 44.5 m. This was in accord with the high Lugeon values in permeability tests. The water levels at Drillhole DA-4 (L = 20 m) at the slope close to the end of the ridge, during drilling and final, were all below the hole bottom. The final water levels in DA-5 (L = 30 m) and DA-6 (L = 20 m) were at depths of 4.5 m and 2.6 m, respectively, roughly equal to the water level elevation of the Devrek River.

Sections below depths of 25 m in Drillhole DA-1 and 40 m in Drillhole DA-2 all indicated low permeabilities of 1 Lu and under. Drillhole DA-3 indicated comparatively high permeabilities of 3 to 10 Lu between depths of 10 and 50 m, while deeper than 50 m, Lu of 1 to 3 were indicated.

iv) Geophysical Prospecting

The result of the survey at the A Layout route shows that there are the 1a layer, a few m thickness, below the surface, the 2a layer, 10~15 m thickness, below the 1a layer, and the 2c or 2b layer below them.

v) Engineering Geology

1. The intake is of the same topography and geology as D Layout, the evaluation being the same, and there is nothing to pose a major problem.

2. The route of the headrace tunnel consists as a whole of a gently sloped mountainland, with rock cover of approximately 60 to 160 m, and cover is thin compared with D Layout. There are more than a dozen gullies crossing the headrace tunnel and all are small, but since rock cover is small in the first place, cover thicknesses becomes less than 100 m where there are gullies. Adding parts of low elevation around the surge tank, approximately one half of the total length of the headrace tunnel has thickness of cover of less than 100 m. It may be considered that there will be some kind of deterioration due to weathering at these sections.

   The tailrace tunnel route is a granodiorite distribution zone and the rock quality at parts which are fresh is generally hard and dense, and a bedrock of about Class c can be expected, but in results of drillhoies investigations made this time, the bedrock at the elevation of the tunnel route was almost all of Class d.

   Hydrothermal alteration is seen prominently at the ground surface of a section of approximately 800 m at the middle part of the headrace tunnel route and approximately 800 m in the vicinity of the surge tank, and it is considered that this hydrothermal alteration has caused extreme deterioration of the bedrock at the elevation of the tunnel route.

Fissures are generally numerous, the spacings being 1 to 10 cm in boring cores, and RQD is less than 20% (Very Bad). The only prominent fault intersecting the tailrace tunnel is the one of strike of northeast-southwest and dip of 85° to the northwest diagonally crossing the tunnel approximately 800 m toward the dam side from the surge tank, but in aerial photographs there are seen 3 lineaments of north-south strike diagonally crossing the tunnel, and it is possible that these comprise fractured zones in the bedrock.

To summarize the above results, deterioration of bedrock is seen at sections of thin rock cover, parts of hydrothermal alteration, the fracture zone around the fault, and parts where cracks are developed, and it is thought the length of these sections together amounts to more than one half of the entire length.

According to the results of permeability tests, permeability is low at 3 Lugeons and under, and except for around the fault, it is considered there will be no great springing of water at depths where cover is thick.

3. The surge tank is planned in the vicinity of EL 450 m on a ridge extending north-south and sloping gently to the north. This location is in a hydrothermally altered granodiorite zone, and according to the results of Drillhole DA-3 provided at this spot, weathering and alteration has occurred down to the hole bottom at a depth of 70 m, and the bedrock is Class d. Fissures

are developed and there are many sections where RQD is zero % (Very Bad). The bedrock is strongly weathered so that it will be necessary to exercise care in carrying out excavation.

In permeability tests, permeability of more than 3 Lu was indicated down to a depth to 50 m, and the final drillhole water level was deep down at 45.5 m so that it will be necessary to be careful about water springing during excavation.

4. The penstock is planned at a ridge running north-south from the surge tank and sloping gently to the north. The penstock will go in a direction to the north approximately 400 m as a tunnel type and then approximately 500 m as a surface type. The tunnel section will have a cover of approximately 80 m at maximum in the vicinity of the surge tank with this cover becoming slightly thinner going towards the north. The inclination of the ridge at the projected surface type site is approximately 10 to 20°. This place is in a distribution zone of granodiorite which has been subjected to hydrothermal alteration, and according to the results at DA-4 drilled here, there have been strong weathering and alteration down to the hole bottom at a depth of 20 m, and the bedrock is Class e. The bed rock is weathered to the depth of about 20 m. Since rocks are partially strongly deteriorated and weakened, it will be necessary to be careful about tunnel excavation and penstock settlement.

5. The powerhouse is planned as an open air type at the end of this ridge. This location is at the boundary between river-bed deposits and the distribution zone of granodiorite subjected to hydrothermal alteration, and according to the

results at DA-5 drilled here, there is anchoring on rock at a depth of 5 m, and the bedrock is Class c to d.

6. The tailrace of open canal type is of the same topography and geology as in D Layout, and the evaluation is also the same.

7.2 Materials

Material investigations made related to this Project are those by the EIE (1964), the DSI (1991) and the Survey Team and DSI carried out this time. In the latter investigation, surface reconnaissances of sites where collecting materials was possible were carried out by the Survey Team and the DSI, and laboratory tests on samples collected from the various sites by the DSI. These laboratory tests were performed on impervious materials (core materials), pervious materials (concrete aggregates), and rock materials. The results of comprehensive analyses of these is described here. The locations of the materials investigation sites are shown in Figure 7-17 and 7-18 and the names of the individual investigation sites and the relations of the varieties of materials which were the objects of investigations in Table 7-11.

Table 7-11 Investigation Areas for Construction Materials

Name of Investigation Area	Kind of Construction Material
A_1, A_2, A_3, B, E, H, I. L, M, N	Impervious Material (Core Material)
D, F, K	Semi-Pervious Material (Filter Material)
C. G	Pervious Material (Concrete Aggregate)
T-1, T-2, T-3, T-4, T-5, Q	Rock and Ripra Material

The embankment volume of dam and concrete total volume of the project are as follows.

Table 7-12 Volume of Dam Embankment and Concrete

Work Item	Unit	Volume
Embankment		
Impervious Core	m^3	779,000
Fine Filter	m^3	328,000
Coarse Filter	m^3	407,000
Rockfill	m^3	3,132,000
Riprap	m^3	132,000
Concrete	m^3	107,000

7.2.1 Impervious Material (Core Material)

The total quantity of impervious core material to be used for the dam embankment is approximately 800 x 10^3 m^3.

There were 9 sites within an area of a radius of 6 to 7 km upstream and downstream with the dam site at the center selected in the Master Plan as candidate sites for impervious materials (core materials). All of the investigation sites are topographically gently sloped or float land where geologically, topsoil, terrace deposits and talus deposits are distributed comparatively thickly, including strongly weathered parts of the basement rock.

The existing results of investigations and tests concerning impervious core materials are as followings.

(1) Existing Test Results

Soil (core) material tests have been carried out on the nine sites of A, B,E, F, H, I, L, M, and N. The test results are given in Table 7-13 and the evaluations are as follows:

- Specific gravities are satisfactory with the range being 2.57 to 2.85 for all sites.

- The results of compaction tests are satisfactory with fairly high values of dry density at optimum water contents in a range of 1.640 to 2.200 t/m^3 for all sites.

- Optimum water content values were of general nature, being 8.2 to 21.7% for all sites.

- Regarding liquid limits (LL), plastic limits (PL), and plasticity indices (PI), although slightly high values were seen for liquid limits (LL) for parts of the soil materials of the I, L, M, and N sites, the greater parts were included in the GC and SG categories considered to be the optimum soil classifications as impervious soil materials on the plasticity diagram, and thus are adequate as soil materials (core material) for a general fill-type dam.

- Natural grain-size distributions are more or less satisfactory for soil materials of the A, B, E, F, and H sites and, moreover, since these are with in the standard grain-size distribution range, there will be problem.

  A trend is seen for fine-grained portions to be slightly high for the soil materials of the I, L, M, and N sites and it will be necessary for some amount of gradation adjustments to be made when using.

(2) Additional Investigation

Of the core material sites investigated in the past, the A and F sites indicted in Figure 7-17 may be mentioned as candidate sites which are advantageous overall in terms of quality-wise, quantity-wise and distance-wise conditions as well as does not necessary expropriation for obtaining materials.

Additional investigations were carried out on these two sites. The contents of the investigations were as follows.

- Outdoor Investigations
  - Geological mapping: preparation of geological maps using 1/5,000 topographical maps
  - Test pit investigation: preparation of geological logs for three pits at each site, collection of data, depth of each pit: 5 m

- Laboratory Tests

  Addition of the items below to the items already tested:
  - Water content measurement: natural water content
  - Gradation measurement: Grain-size distribution of 0.074 mm and finer
  - Permeability test: Coefficient of permeability at optimum water content
  - Triaxial compression test: Cohesive force (c) and angle of internal friction (ϕ)
  - Number of tests/measurements: the number of each of the above-mentioned tests measurements to be not less than three times

The location and geological logs for pits are shown in Figure 7-19 and 7-20.

The laboratory test results are indicated in Table 7-14, Figure 7-21, 7-22, 7-23, 7-24 and 7-25 respectively. Summaries of the test results are given below.

- Specific gravities are satisfactory with the range being 2.73 to 2.85.

- The results of compaction tests are satisfactory following that dry densities at optimum water contents are in the range of 1.73 to 2.06 gf/cm^3 to indicate high values.

- Regarding optimum water content obtained from proctor compaction tests, they are from 8.6 to 17.5%, and these are values in a general range.

- Regarding liquid limits (LL), plastic limits (PL), and plasticity indices (PI), slightly low values of 7.0 to 15.1 of plastic limits (PI) were seen for all test results.

  Paying attention to the test results of A1 material is necessary, because two samples were classified as silty sand (SM), and two (2) out of three (3) samples could not be tested.

  Concerning with the material of A2, A3 and F sites, plasticity indices (PI) value from 7.1 to 15.1, and soil were classified clayey sand (SC) and inorganic clay (CL) categories considered to be the optimum soil classifications as impervious core material.

- With respect to grain-size distribution, test results were satisfactory for impervious core material, since these were within the standard grain-size distribution, with the quantities of #4 (4.75 mm) and under being from 60.0 to 95.9% and #200 (0.075 mm) and under from 16.0 to 60.8%.

- Triaxial shear test were performed by two (2) methods, undrained-unconsolidated (UU) and drained-unconsolidated (DU), and cohesion and angle of internal friction were from 0.25 kgf/cm^2 to 1.10 kgf/cm^2 and from 13° to 35° for UU test, and from 0 to 0.43 kgf/cm^2 and from 18° to 38° for DU test respectively.

- Permeability tests were performed at the optimum water content obtained by proctor compaction test with each three (3) samples from A1, A2, A3 and F sites. As a result, samples from A1 site had from 4.32 x 10^{-4} cm/s to

4.4×10^{-3} cm/s, permeability was rather high. Considering permeability test results and Atterbery limits tests, A1 site is not necessarily suitable.

Test results of A2, A3 and F sites material were satisfactory as the impermeable core material indicating the permeability coefficient from 1.7×10^{-4} cm/s to 2×10^{-7} cm/s.

(3) Evaluation

The assessments of the above test results are as follows.

- A1 soil material is a silty sand (SM) including clayey sand (SC) according to the unified soil classification. The grain-size distribution, #200 (0.075 mm) and under from 16.0 to 36.1% is suitable for impervious core material and dry density from 1.91 to 2.06 gf/cm^2 also satisfactory, on the other hand permeability coefficient is high.

- Regarding A2 soil material, the quality is more uniform comparing with other proposed areas and suitable for impervious core material, in addition it will be possible to transport the material from borrow area to dam embankment directly. The soil is classified as clayey sand (SC), gradation -#200 (0.075 mm)-and under is from 26.4 to 49.7%, dry density is from 1.83 to 1.99 fg/cm^3, angle of internal friction is from 13° to 19° (UU), permeability coefficient at optimum water content is from 1.69×10^{-4} to 2.2×10.

- A3 soil material is a clayey sand (SC) containing in organic clay (CL) according to the unified soils classification. Grading is rather fine with #200 (0.074 mm) and under from 32.8% to 56.9%. Dry density is satisfactory with the range being 1.80 to 1.94 gf/cm^3. The soil is suitable as impervious core material with above mentioned values and angle of internal friction from 15° to 27° and from 1.7×10^{-5} to 4×10^{-7} cm/s of permeability coefficient.

- As for F soil material, it is classified as inorganic clay (CL) including clayey sand (SC).

The soil feature varies widely with the grading #200 (0.074) and under from 18.0 to 60.8% and the dry density from 1.73 to 2.09 gf/cm^3. The angle of internal friction is from 15° to 26°. The permeability coefficient is low with from 1.48×10^{-4} to 2.0×10^{-7} cm/s. The soil is suitable as impervious core material but hauling distance is longer comparing with other proposed borrow areas and a bridge will be necessary for transportation of the material to dam site.

A1 site and F site are not necessarily suitable for the impermeable core material source, because the permeability coefficient of laboratory test result on former site is high and longer distance transportation and a bridge construction are inevitable for latter site material.

It will be possible to excavate 20~30 m deep at A2 site, considering the result of test pit log, geographical features (slope of 1:6) and origin of the soil. Consequently 800×10^3 m^3 of soil material which is equal to impervious core embankment volume may be obtained. In addition it is possible to obtain 600×10^3 m^3 of soil material from A3 site on condition that excavation to 10 m depth.

The laboratory test results show that the soil both at A2, A3 site are suitable as impermeable core material. It is recommended that to arrange A2 site as main borrow area and A3 site as a reserve area on account of distance to dam and probable volume.

Additional investigation boring would be desirable for making sure the available volume of soil material on A2 and A3 site.

For making the comparative study, the laboratory tests of impervious core material on the site of L, M and N where there was no additional investigation at this stage must be made on the stage of final design.

7.2.2 Filter Material

The quantity of fine and course filter material to be used for dam embankment would be approximately 330,000 m^3 and 410,000 m^3 respectively.

The results of laboratory tests of a materials from the D, F, and K sites investigated at master plan stage considering semi-pervious materials had indicated that the contents of fines material (mainly clay) were slightly high, and it had been pointed out that attention should be paid when D.F.K. sites materials are to be used as filter one those materials include some quantity requiring graduation adjustment. In the additional investigations made this time, laboratory tests were not conducted, with only field reconnaissances made.

As a result, on comprehensive judgement of quality-wise and quantity-wise conditions and distance-wise conditions, it was revealed that it would be most appropriate to use the river-bed sand-gravel from the C site investigated as concrete aggregate for fine filter materials, and fine rock material at site Q and rock excavation much obtained from the vicinity of the dam for coarse filter materials.

The riverbed deposit in C site is suitable for fine filter material with its quality-wise and gradation-wise conditions as the laboratory test result is described in close 7.2.3 Pervious Material (Concrete Aggregate). The quantity available at C site is satisfactory, because the area of riverbed deposit (GI) is more wider than expected at master plan stage according to the site reconnaissance result as shown in Figure 7-19.

7.2.3 Pervious Materials (Concrete Aggregates)

Regarding pervious materials (concrete aggregates) investigations were carried out at the C and G sites on river-=bed gravel of the Devrek River. Site C is located at the midstream part inside the reservoir area, and Site G in the vicinity of the projected outlet site.

(1) Existing Test Results

Concrete aggregate tests have been performed on samples collected from the two locations of the C and G sites and the test result is shown in Table 7-13. The evaluations for fine and coarse aggregates are as follows:

Fine Aggregates

- Specific gravities are from 2.59 to 2.71 and these values are in the standard range for concrete aggregates.

- Unit weights are from 1.65 to 1.82 ton/m^3 and are in the standard range for concrete aggregates.

- Absorption ranges are from 0.8 to 2.7% to be in the range for aggregate in general of 0.3 to 3.0%, and there will be no problem.

- The results of washing tests of aggregates (content of soil particles 0.074 mm and under) are 1.64 to 9.31%, and since the standard value for concrete aggregate in case of fine aggregate is put at 5% and under, this is a slightly high value.

- Soundness results are from 5.8 to 14.2%, and since the standard values are stipulated to be 10% and under in case of fine aggregates the result are slightly high.

- Gradations are approximately within the range of the standard gradation for fine aggregate and are satisfactory. Fineness moduli (F.M.) are 2.54 to 3.40, which are standard values.

Coarse Aggregates

- Specific gravities are from 2.67 to 2.75 and these values are in the standard range for concrete aggregates.

- Unit weights are from 1.91 to 2.00 ton/m^3 and are in the standard range for concrete aggregates.

- Absorption ranges are from 0.6 to 0.9% to be in the range for aggregates in general of 0.4 to 3.0%, and there will be no problem.

- The results of washing tests of aggregates (content of soil particles 0.074 mm and under) are 0.06 to 0.40%, and are satisfactory.

- Soundness results are from 10.8 to 23.3%, and since the standard values are stipulated to be 12% and under in case of coarse aggregates, the results are slightly too high.

- Gradations are satisfactory, being more or less in the range of standard gradation for coarse aggregate. Fineness moduli (F.M.) are 7.54 to 7.80, and are standard values.

(2) Additional Investigation

With respect to concrete aggregate investigations, gradation tests and physical property tests had already been performed on samples collected from the C and G sites indicated in Table 7-13.

The additional investigation was carried out to confirm quality-wise conditions including fine filter and physical property-wise conditions. The contents of the investigations were as follows:

- Field Investigations
  - Geological mapping: preparation of geological maps using 1/5,000 topographical maps
  - Sampling: collection of samples as concrete aggregates from representative locations of both sites

- Laboratory Tests

  Addition of the items below to the items already tested
  - Abrasion tests
  - Alkali-aggregate reaction test
  - Number of tests: not less than three times for each of the above two tests
  - Test specifications: technical specifications for the above two tests to be according to Turkish standards and ASTM as a rule

The results of filed investigations such as geological map and test pit section are shown in Figure 7-19 and 7-20 respectively. The laboratory test results are given in Table 7-14, 7-15, and Figure 7-26 in shown grain-size distribution.

(3) Evaluation

According to investigation results, although there are slight drawbacks in results of washing tests and soundness tests with respect to fine aggregates and in results of soundness tests with respect to coarse aggregates, it has been found that there are no problems regarding other physical properties and gradation. Therefore, in case of using these materials, it is considered they can serve as concrete aggregates possessing adequate suitability if washing of aggregates is carefully done.

Upon overall judgement of quality-wise and quantity-wise conditions and distance-wise conditions, it was shown that using the river-bed sand-gravel available at Site G will be the most suitable.

Abrasion tests and alkali aggregate reaction test are not available yet,

7.2.4 Rock Materials

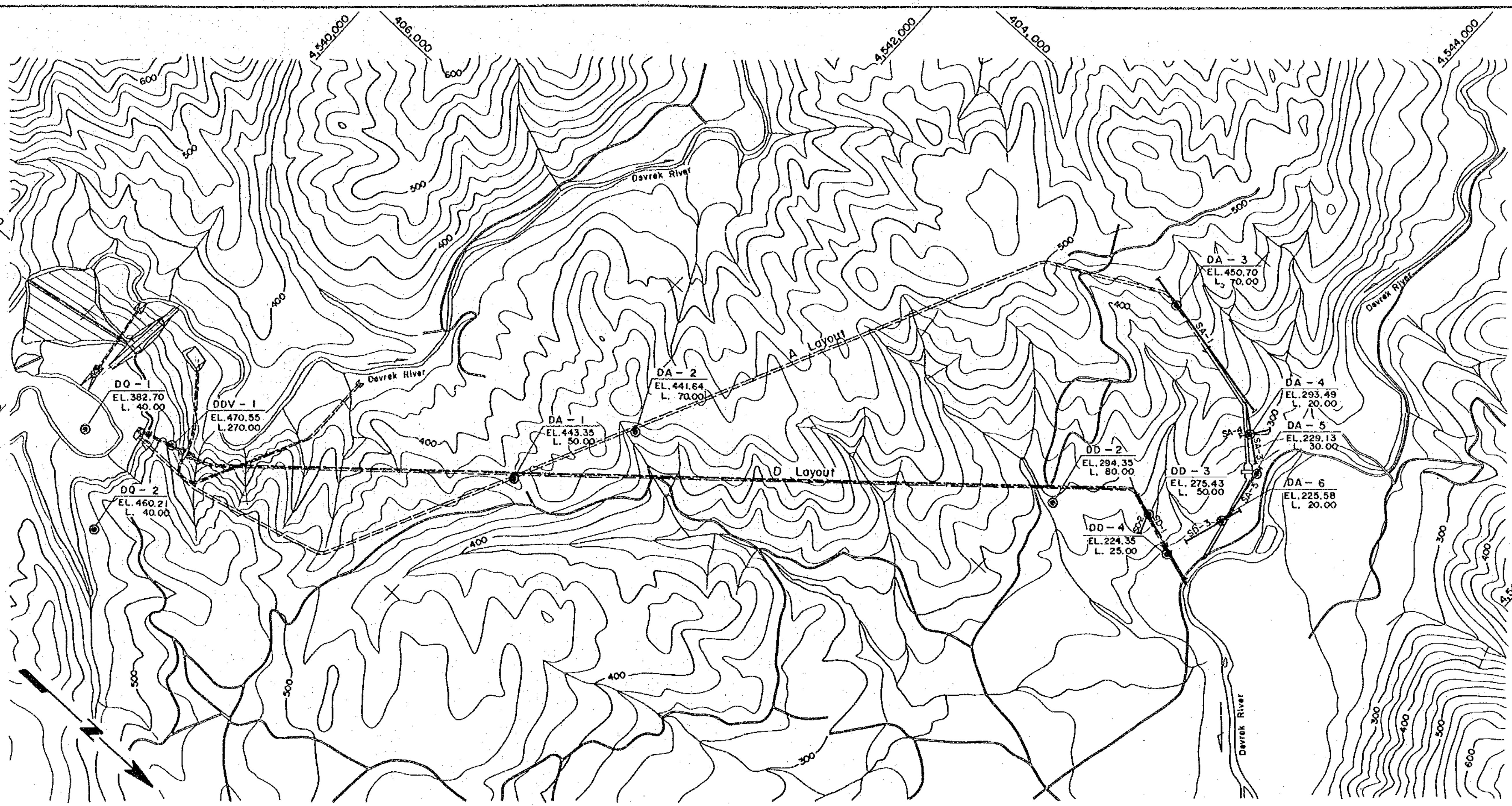
With regard to rock materials and rip-rap materials, investigations had already been carried out on the five sites of T1, T2, T3, T4, and T5. Those test result are given in Table 7-13. One of the sites among the above where investigations were made on the granodiorite. Other were investigation sites on the limestone. Regarding samples obtained from these sites, laboratory tests have ben conducted to judge the applicabilities rock material, and the results of those tests have indicated there will be no problem from the standpoint of quality. However, the limestone in the reservoir has the drawback that it is far away from the dam site, the hauling distance being approximately 9 to 11 k.

Therefore, Site Q (the islet-like mountain body immediately upstream of the damsite and the mountain body at the opposite right-bank side) was newly selected, and additional investigations were made. The results of the investigations indicate there will be no problem for using this rock material.

The location of boring, geological log of drillholes and laboratory test results are given in Figure 7-1, 7-15 and Table 7-16.

On overall judgement of quality-wise and quantity-wise conditions and distance-wise conditions, it was made clear that using the material available at Site Q, Granodiorite, would be most suitable for rock materials and using the material available at Site T, Limestone, would be most suitable for rip-rap materials.

Furthermore, abrasion test and alkali-aggregate reaction test are not available yet, these tests would be desirable to finish before definite design.



Drillhole No. (Temporary)	Location	Length (m)	Elevation (m)	Coordinate X	Coordinate Y	Water Measurement	Lugeon Test
DA-1	Headrace tunnel route - Case A	50.0	442.35	407,207.52	4,541,060.37	o	25.0 - 50.0 m
DA-2	Headrace tunnel route - Case A	70.0	441.64	406,637.66	4,542,341.05	o	40.0 - 70.0 m
DA-3	Surge tank site - Case A	70.0	450.70	404,411.42	4,543,853.42	o	10.0 - 70.0 m
DA-4	Penstock route - Case A	20.0	293.49	404,661.93	4,544,544.84	o	-
DA-5	Powerhouse site - Case A	30.0	229.13	404,778.36	4,544,698.57	o	-
DA-6	Tailrace canal route - Case A	20.0	225.58	405,057.20	4,544,721.52	o	-
DDV-1	Powerhouse site - Case D	270.0	470.55	408,206.77	4,540,749.22	o	170.0 - 270.0 m
DD-2	Tailrace tunnel route - Case D	80.0	294.35	405,550.55	4,545,050.00	o	50.0 - 80.0 m
DD-3	Tailrace tunnel route - Case D	50.0	275.43	405,264.17	4,544,050.41	o	-
DD-4	Tailrace canal route - Case D	25.0	224.35	405,346.89	4,544,646.09	o	-
DQ-1	Quarry site	40.0	382.70	408,426.05	4,540,366.51	o	-
DQ-2	Quarry site	40.0	460.21	408,750.87	4,540,723.95	o	-

Legend

DA - 1
EL.443.35
L. 50.00 Drillholes

SA-I Seismic Prospecting

0 1000m

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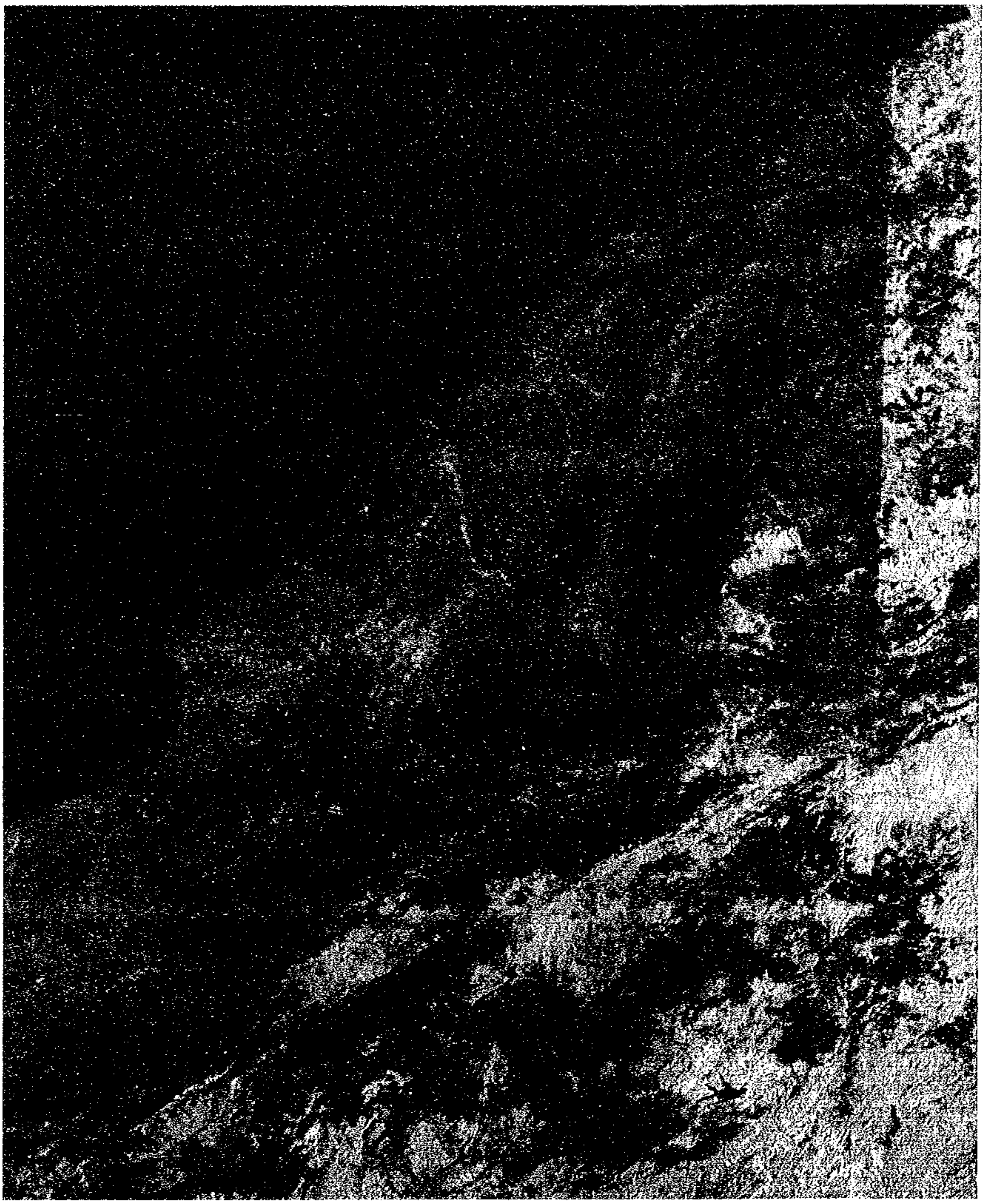
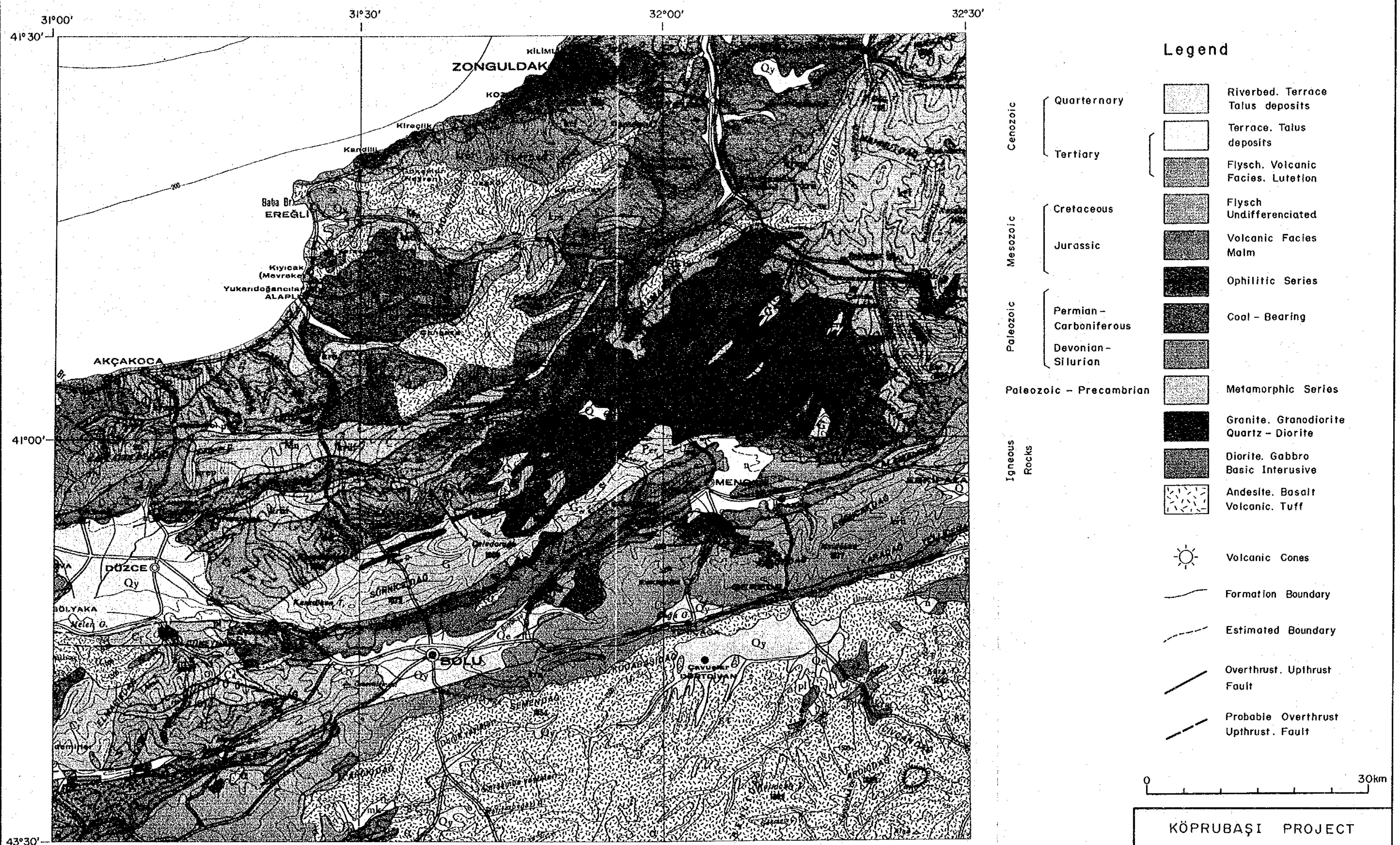
LOCATION MAP OF ADDITIONAL INVESTIGATIONS

Figure 7-1



Landsat Image of Project Area



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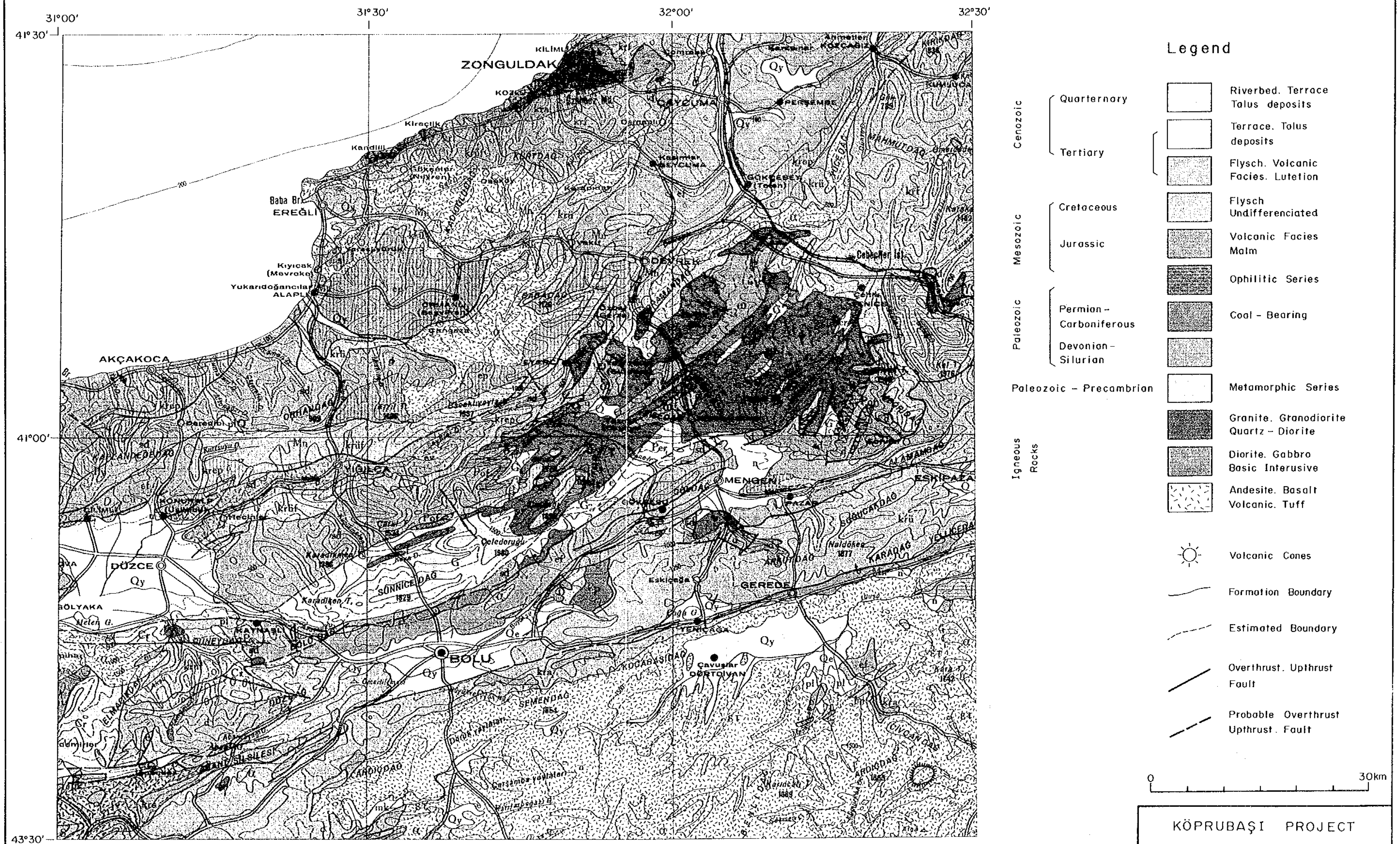
Regional Geological Plan

Note : This map is quoted from "Geological Map of Turkey, Zonguldak" by the Institute of Mineral Research and Exploration in 1964.



Note: This map is quoted from "Geological Map of Turkey, Zonguldak" by the Institute of Mineral Research and Exploration in 1964.

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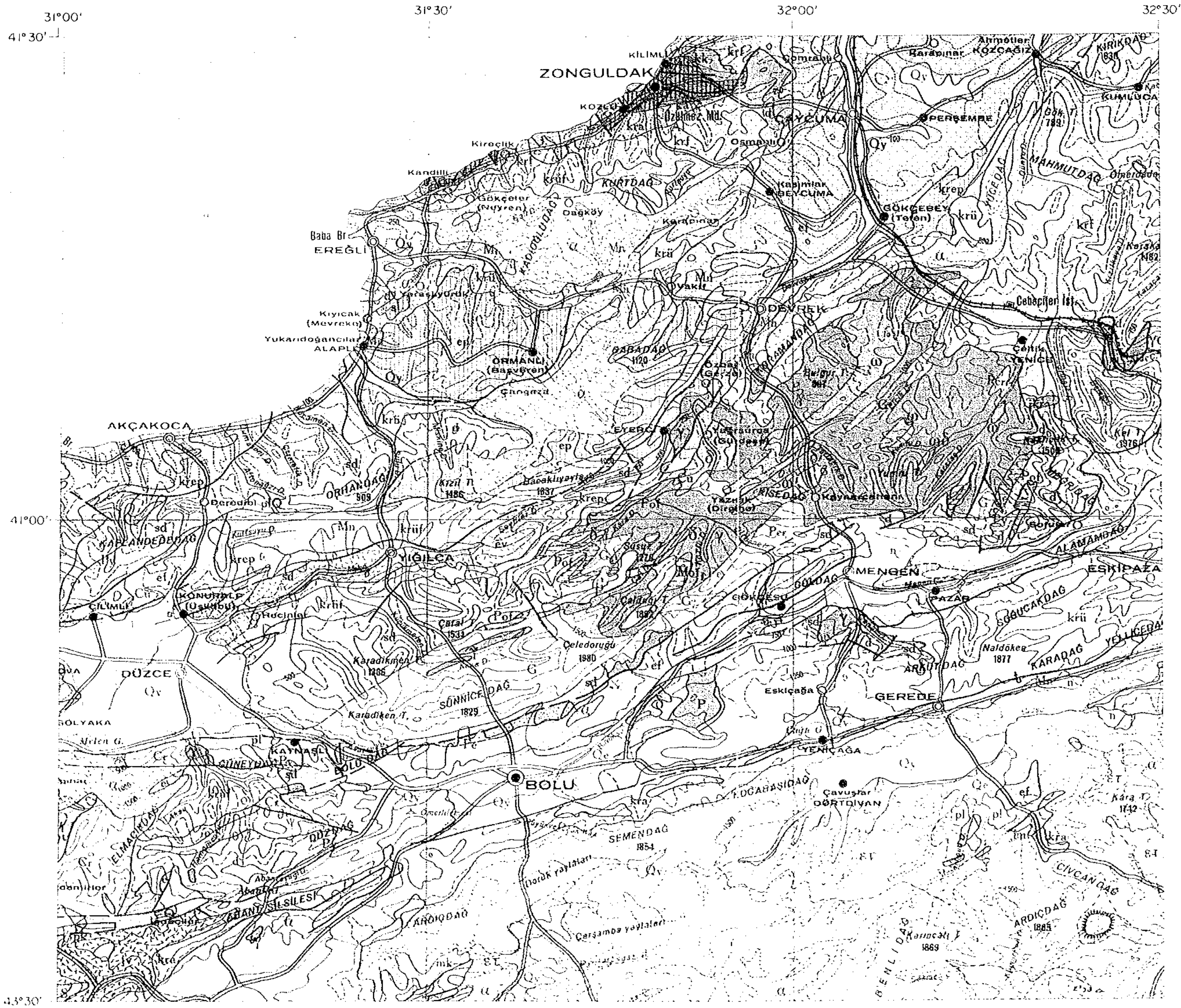
Regional Geological Plan



Legend

Cenozoic	Quarternary	Riverbed. Terrace Talus deposits
		Terrace. Talus deposits
	Tertiary	Flysch. Volcanic Facies. Lutetion
Mesozoic	Cretaceous	Flysch Undifferenciated
	Jurassic	Volcanic Facies Malm
		Ophilitic Series
Paleozoic	Permian – Carboniferous	Coal – Bearing
	Devonian – Silurian	
Paleozoic – Precambrian		Metamorphic Series
Igneous Rocks		Granite. Granodiorite Quartz – Diorite
		Diorite. Gabbro Basic Intrusive
		Andesite. Basalt Volcanic. Tuff

Volcanic Cones

Formation Boundary

Estimated Boundary

Overthrust Upthrust Fault

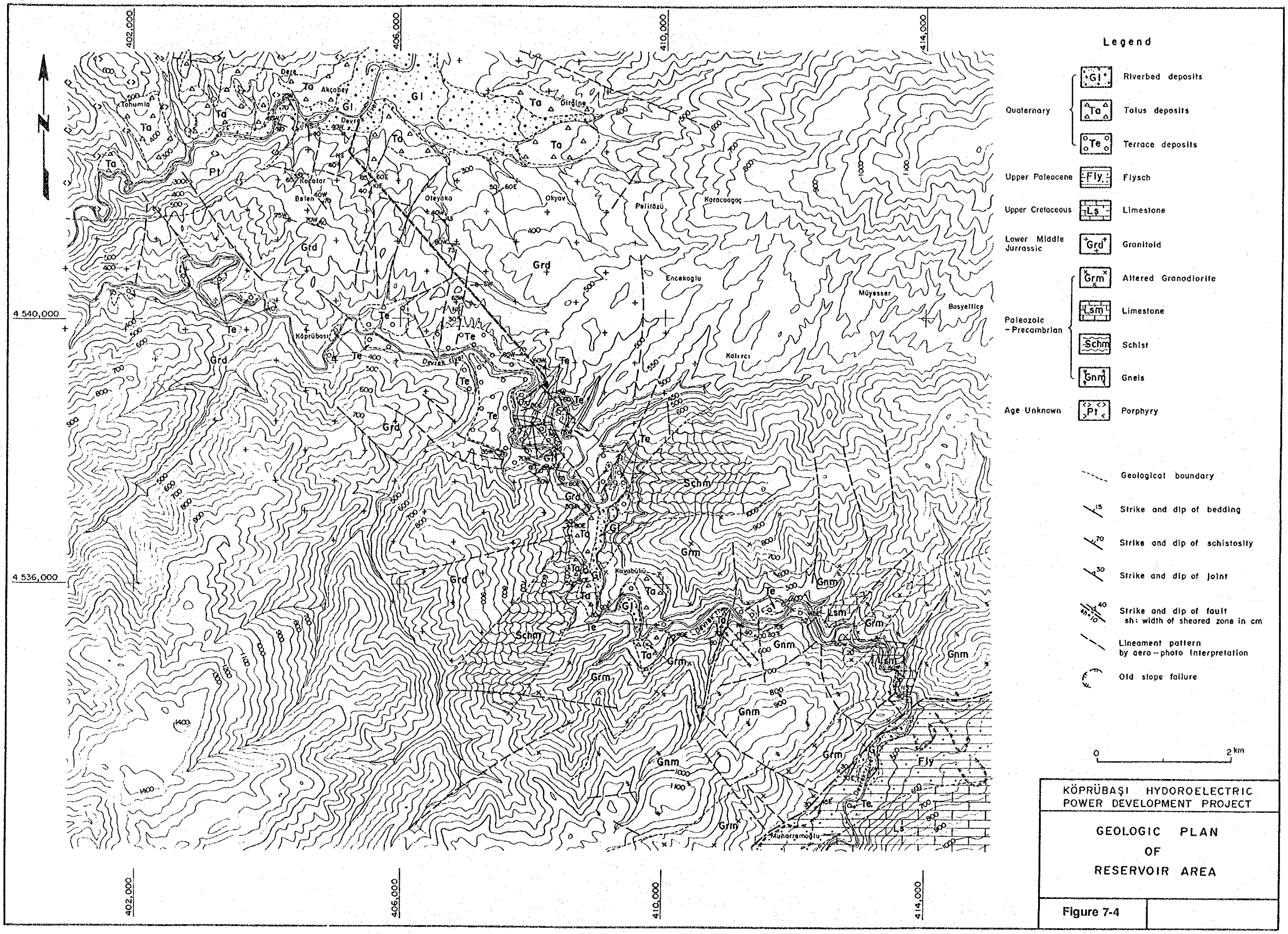
Probable Overthrust Upthrust. Fault

0 30km

Note: This map is quoted from "Geological Map of Turkey, Zonguldak" by the Institute of Mineral Research and Exploration in 1964

KÖPRUBAŞI PROJECT

Regional Geological Plan



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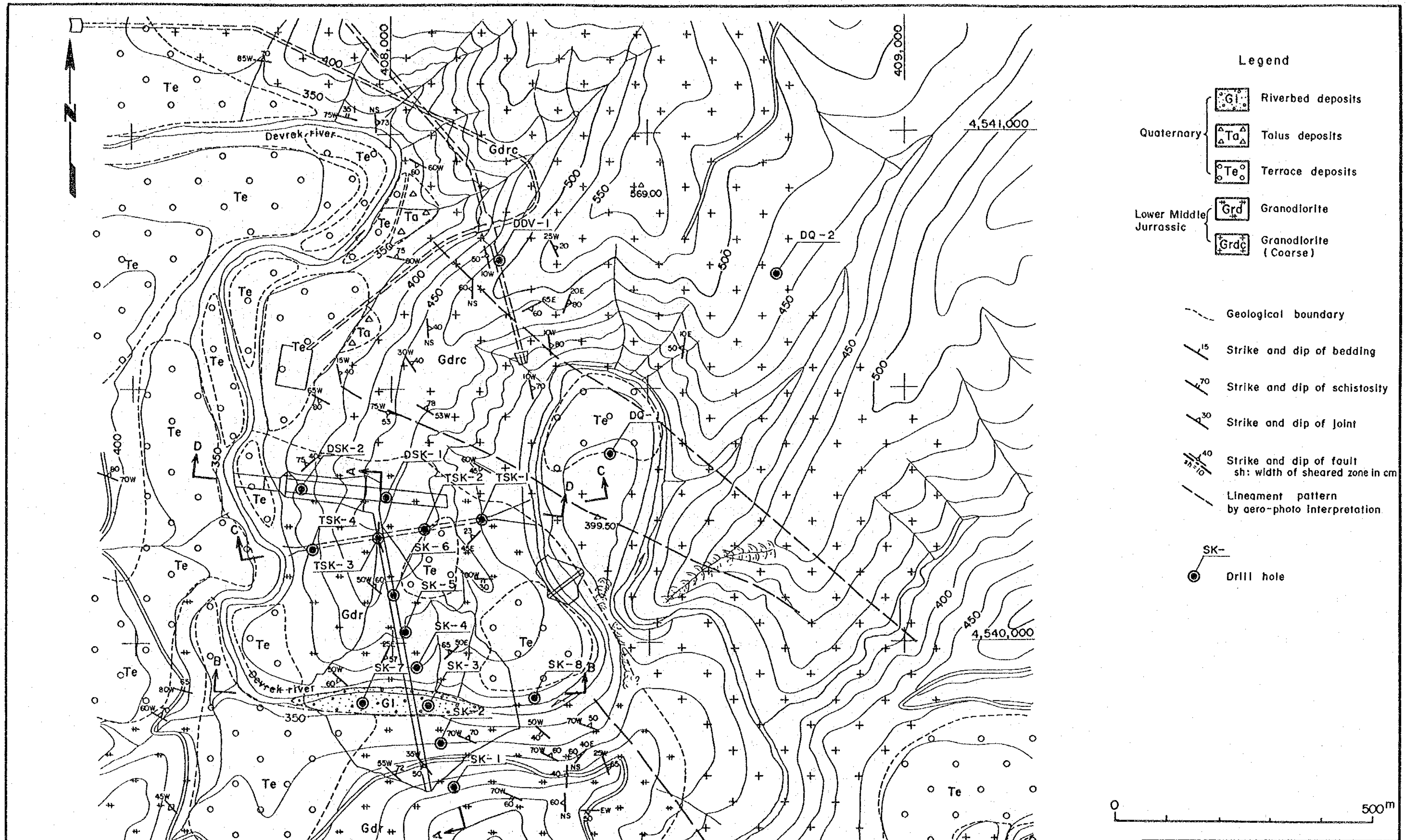
GEOLOGIC PLAN OF RESERVOIR AREA

Figure 7-4



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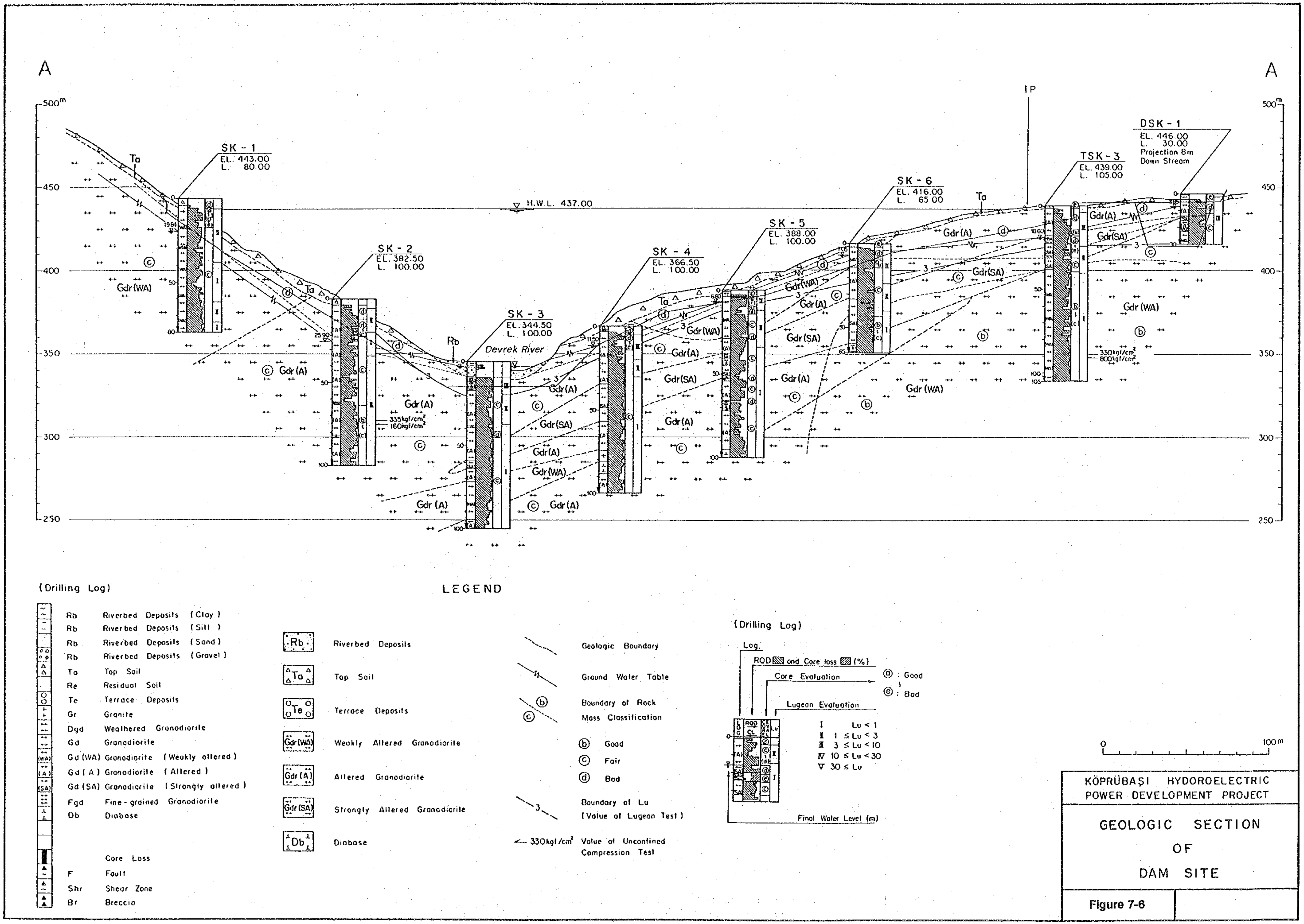
GEOLOGIC PLAN OF DAM SITE

Figure 7-5



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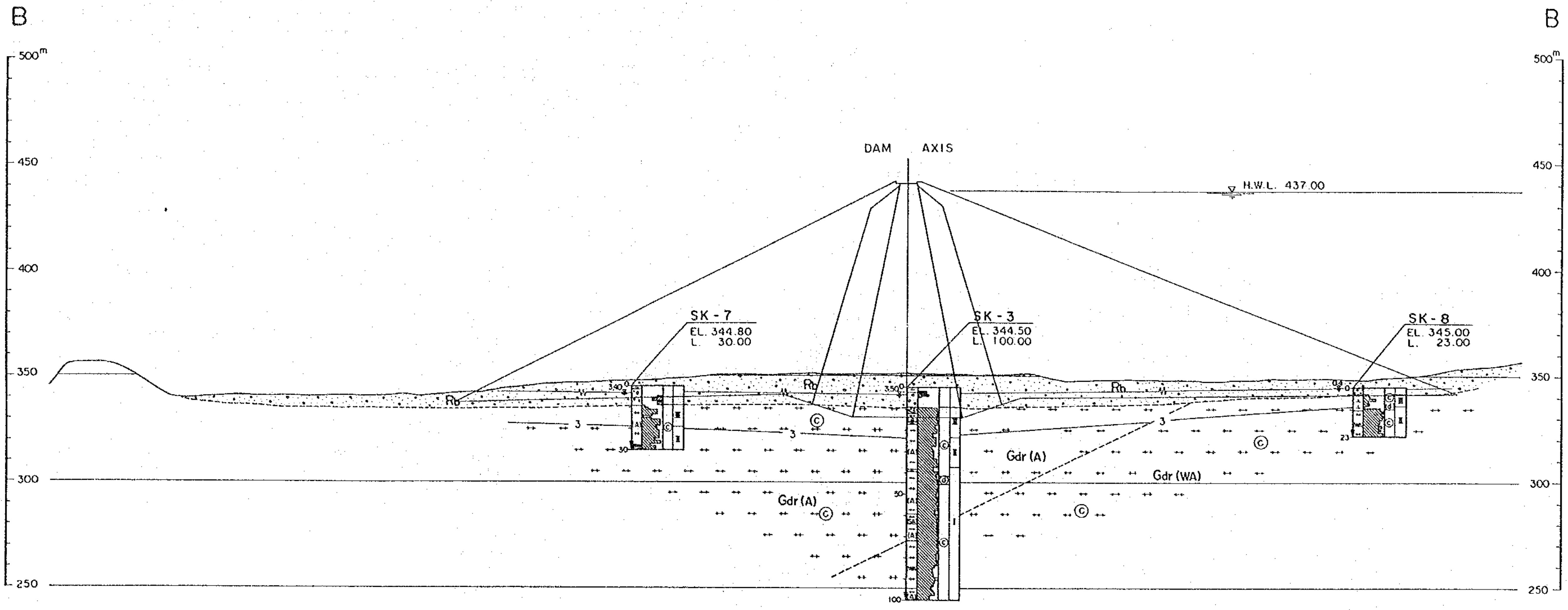
GEOLOGIC SECTION OF DAM SITE

Figure 7-6



(Drilling Log)

Rb	Riverbed Deposits (Clay)
Rb	Riverbed Deposits (Silt)
Rb	Riverbed Deposits (Sand)
Rb	Riverbed Deposits (Gravel)
Ta	Top Soil
Re	Residual Soil
Te	Terrace Deposits
Gr	Granite
Dgd	Weathered Granodiorite
Gd	Granodiorite
Gd (WA)	Granodiorite (Weakly altered)
Gd (A)	Granodiorite (Altered)
Gd (SA)	Granodiorite (Strongly altered)
Fgd	Fine-grained Granodiorite
Db	Diabase
	Core Loss
F	Fault
Shr	Shear Zone
Br	Breccia

LEGEND

Rb	Riverbed Deposits
Ta	Top Soil
Te	Terrace Deposits
Gdr (WA)	Weakly Altered Granodiorite
Gdr (A)	Altered Granodiorite
Gdr (SA)	Strongly Altered Granodiorite
Db	Diabase

Geologic Boundary

Ground Water Table

Boundary of Rock Mass Classification

- ⓑ Good
- ⓒ Fair
- ⓓ Bad

Boundary of Lu (Value of Lugeon Test)

(Drilling Log)

Log.

RQD and Core loss (%)

Core Evaluation — ⓐ : Good ~ ⓔ : Bad

Lugeon Evaluation

- I Lu < 1
- II 1 ≤ Lu < 3
- III 3 ≤ Lu < 10
- IV 10 ≤ Lu < 30
- V 30 ≤ Lu

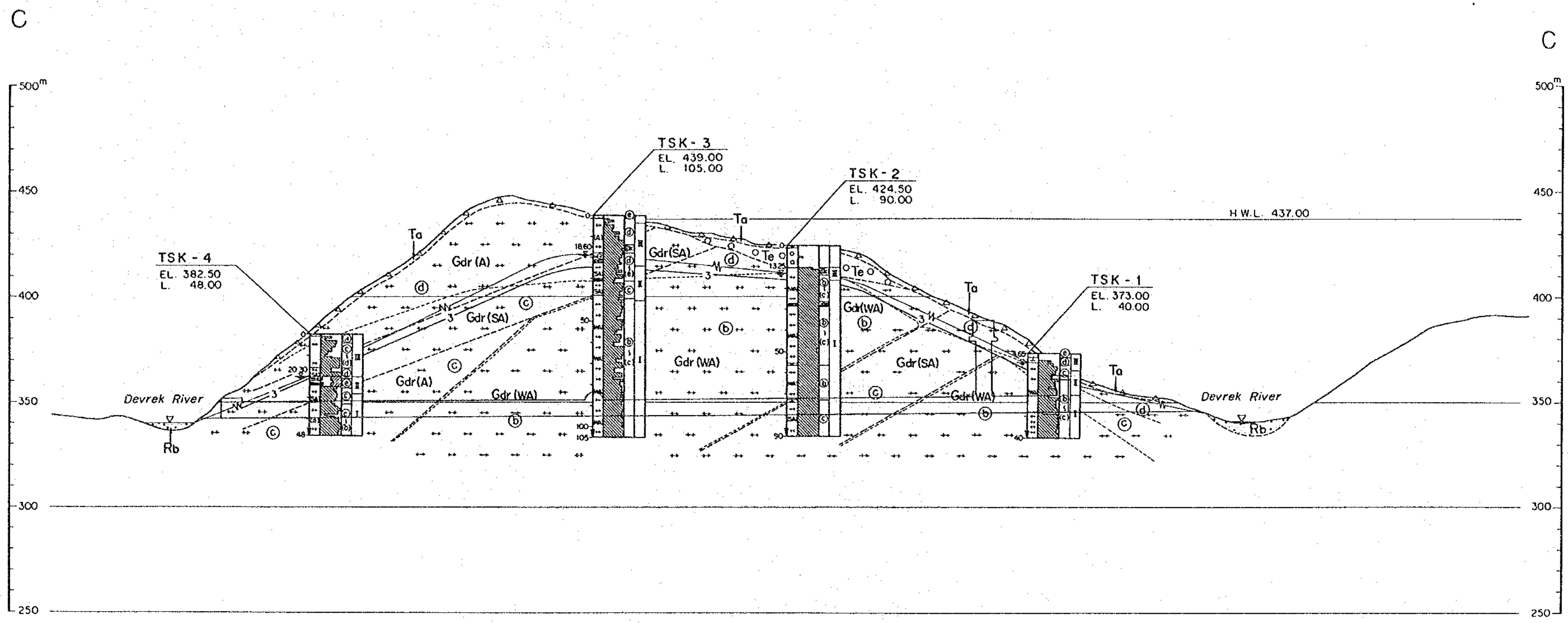
Final Water Level (m)

0 100m

KÖPRÜBAŞI HYDROELECTRIC POWER DEVELOPMENT PROJECT

GEOLOGIC CROSS-SECTION OF DAM SITE

Figure 7-7



(Drilling Log)

Rb Riverbed Deposits (Clay)
Rb Riverbed Deposits (Silt)
Rb Riverbed Deposits (Sand)
Rb Riverbed Deposits (Gravel)
Ta Top Soil
Re Residual Soil
Te Terrace Deposits
Gr Granite
Dgd Weathered Granodiorite
Gd Granodiorite
Gd (WA) Granodiorite (Weakly altered)
Gd (A) Granodiorite (Altered)
Gd (SA) Granodiorite (Strongly altered)
Fgd Fine-grained Granodiorite
Db Diabase
Core Loss
F Fault
Shr Shear Zone
Br Breccia

LEGEND

Rb Riverbed Deposits
Ta Top Soil
Te Terrace Deposits
Gdr (WA) Weakly Altered Granodiorite
Gdr (A) Altered Granodiorite
Gdr (SA) Strongly Altered Granodiorite
Db Diabase

Geologic Boundary
Ground Water Table
Boundary of Rock Mass Classification
ⓑ Good
ⓒ Fair
ⓓ Bad
Boundary of Lu (Value of Lugeon Test)

(Drilling Log)

Log.
RQD and Core loss (%)
Core Evaluation ⓐ : Good ~ ⓔ : Bad
Lugeon Evaluation
I Lu < 1
II 1 ≤ Lu < 3
III 3 ≤ Lu < 10
IV 10 ≤ Lu < 30
V 30 ≤ Lu
Final Water Level (m)

0 100m

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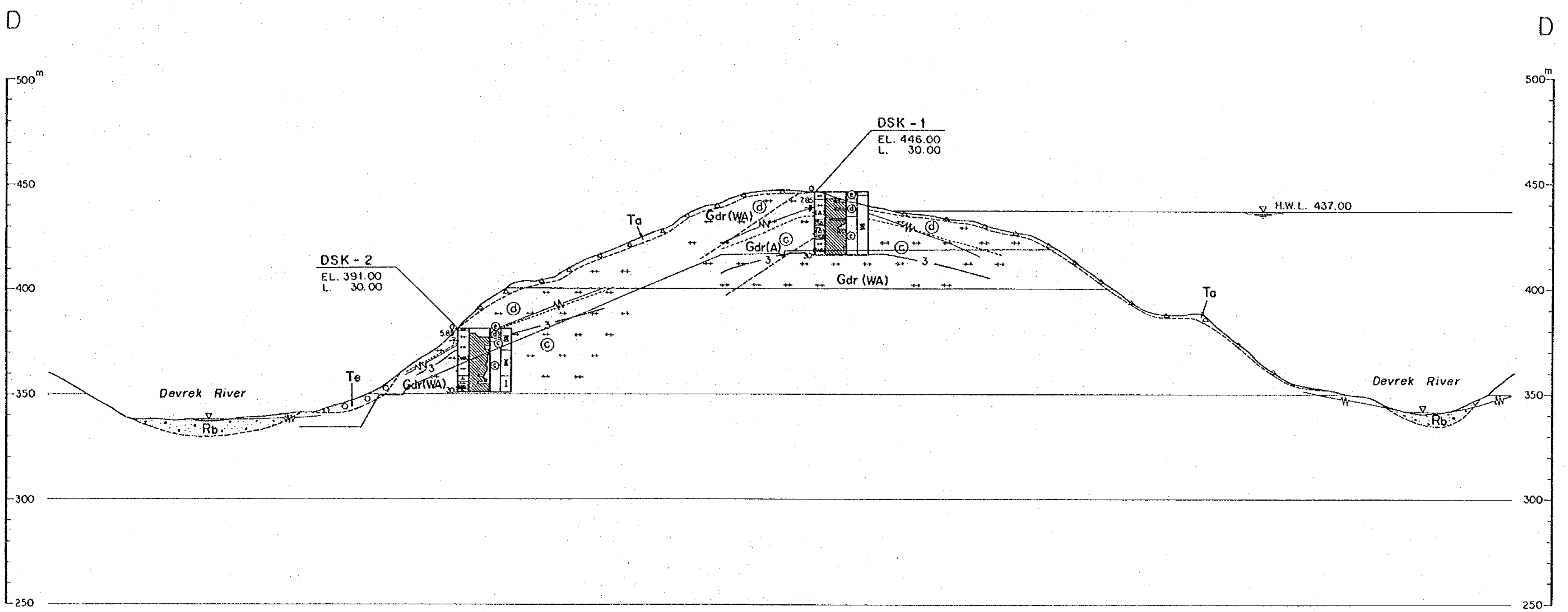
GEOLOGIC SECTION OF DIVERSION TUNNEL

Figure 7-8



(Drilling Log)

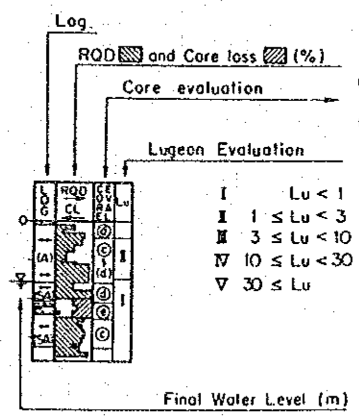
- Rb Riverbed Deposits (Clay)
- Rb Riverbed Deposits (Silt)
- Rb Riverbed Deposits (Sand)
- Rb Riverbed Deposits (Gravel)
- Ta Top Soil
- Re Residual Soil
- Te Terrace Deposits
- Gr Granite
- Dgd Weathered Granodiorite
- Gd Granodiorite
- Gd (WA) Granodiorite (Weakly altered)
- Gd (A) Granodiorite (Altered)
- Gd (SA) Granodiorite (Strongly altered)
- Fgd Fine-grained Granodiorite
- Db Diabase
- Core Loss
- F Fault
- Shr Shear zone
- Br Breccio

LEGEND

- Rb Riverbed Deposits
- Ta Top Soil
- Te Terrace Deposits
- Gdr(WA) Weakly Altered Granodiorite
- Gdr(A) Altered Granodiorite
- Gdr(SA) Strongly Altered Granodiorite
- Db Diabase

- Geologic Boundary
- Ground Water Table
- Boundary of Rock Mass Classification
- ⓑ Good
- ⓒ Fair
- ⓓ Bad
- Boundary of Lu (Value of Lugeon Test)

(Drilling Log)



ⓐ : Good ~ ⓔ : Bad

- I Lu < 1
- II 1 ≤ Lu < 3
- III 3 ≤ Lu < 10
- IV 10 ≤ Lu < 30
- V 30 ≤ Lu

0 — 100m

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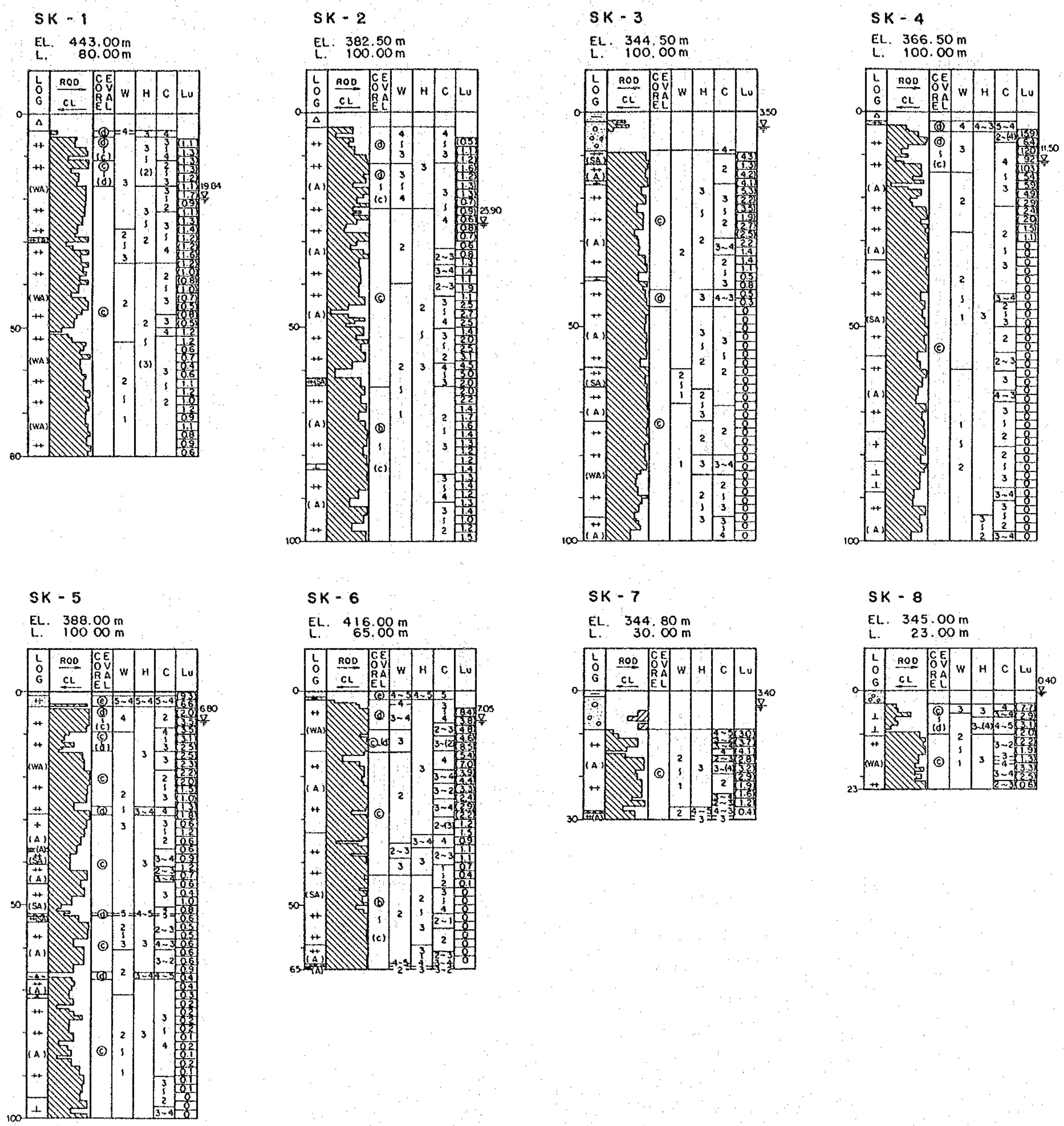
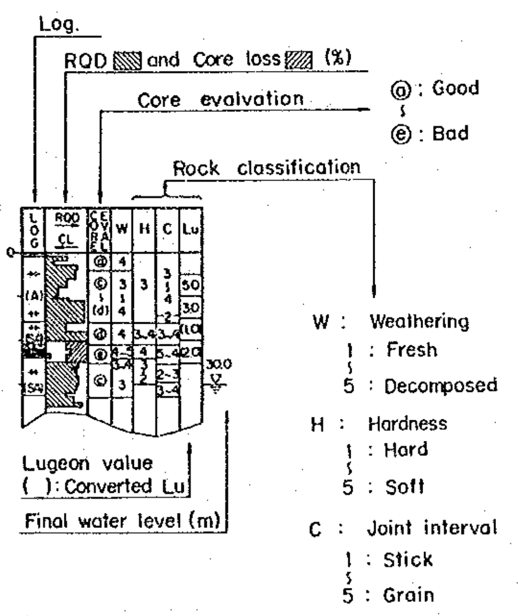
GEOLOGIC SECTION OF SPILLWAY

Figure 7-9



LEGEND

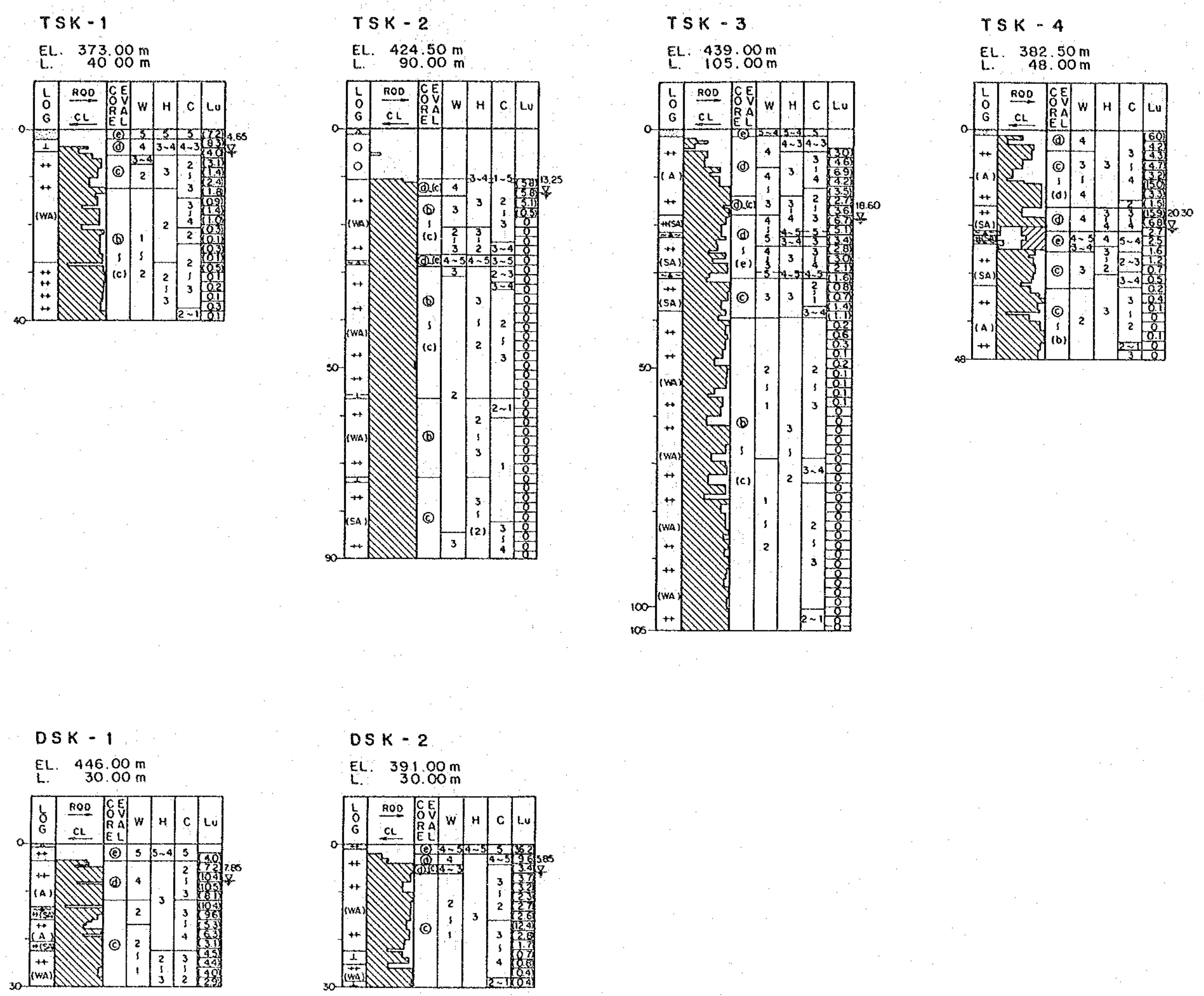
Rb	Riverbed Deposits	(Clay)
Rb	Riverbed Deposits	(Silt)
Rb	Riverbed Deposits	(Sand)
Rb	Riverbed Deposits	(Gravel)
To	Top Soil	
Re	Residual Soil	
Te	Terrace Deposits	
Gr	Granite	
Dgd	Weathered Granodiorite	
Gd	Granodiorite	
Gd (WA)	Granodiorite	(Weakly altered)
Gd (A)	Granodiorite	(Altered)
Gd (SA)	Granodiorite	(Strongly altered)
Fgd	Fine - grained Granodiorite	
Db	Diabase	
	Core Loss	
F	Fault	
Shr	Shear zone	
Br	Breccia	



KÖPRÜBAŞI HYDROELECTRIC POWER DEVELOPMENT PROJECT

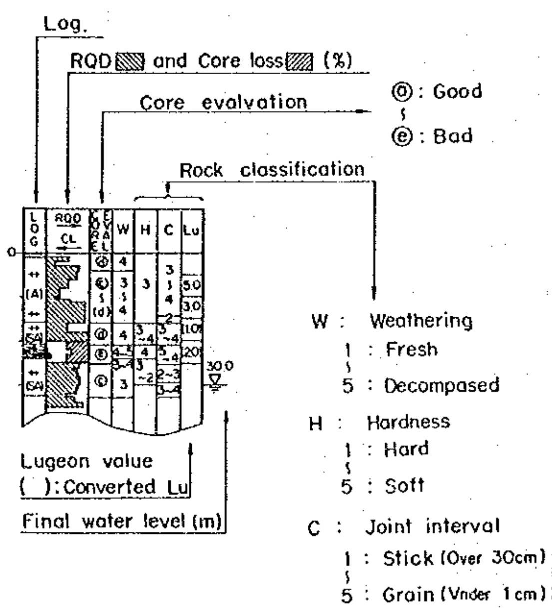
GEOLOGIC LOG OF DRILLHOLES AT DAM SITE

Figure 7-10



LEGEND

Rb	Riverbed Deposits	(Clay)
Rb	Riverbed Deposits	(Silt)
Rb	Riverbed Deposits	(Sand)
Rb	Riverbed Deposits	(Gravel)
Ta	Top Soil	
Re	Residual Soil	
Te	Terrace Deposits	
Gr	Granite	
Dgd	Weathered Granodiorite	
Gd	Granodiorite	
Gd (WA)	Granodiorite	(Weakly altered)
Gd (A)	Granodiorite	(Altered)
Gd (SA)	Granodiorite	(Strongly altered)
Fgd	Fine - grained Granodiorite	
Db	Diabase	
	Core Loss	
F	Fault	
Shr	Shear zone	
Br	Breccia	



KÖPRÜBAŞI HYDROELECTRIC POWER DEVELOPMENT PROJECT

GEOLOGIC LOG OF DRILLHOLES AT DIVERSION TUNNEL AND SPILLWAY

Figure 7-11