DAMS IN HVITA AND THJORSA RIVER BASINS

by

Vladimir Vutsel

United Nations Special Fund

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INTRODUCTION.

Dams of local construction materials and earth and rock fill in particular, became ever more widely used in hydro-technical construction during last 20 years. One of the main reasons is the development of modern machinery for excavation and transport.

The experience gathered in methods of construction and research on earth and rock-fill dams also made it possible to improve the design of these dams resulting in lesser quantities of materials than the ones built in previous years.

It was equally important in the adoption of this type of dams that they do not need so robust foundations requirements as are generally required for concrete gravity dams not to say of arch ones.

In some countries where construction of hydro-power stations began rather early the concrete dams have been built at favourable sites. As the time passed there were fewer such sites left where the foundations conditions would favour the construction of concrete dams. It is also promoted, to a certain degree, the earth and rock-fill dams construction on a wider scale in recent times. It is rather interesting to point out that many experts think that if a problem of building of hydro-electric power stations with high concrete dams, which were constructed during last 20-30 years, had arisen now the earth and rock-fill types would have been chosen as the most economical ones for the above-mentioned reasons.

Iceland abounds in relatively vast undeveloped hydro-resources.

Engineering-geologic conditions at most of the dam sites in the Hvita and Thjorsa river basins make it possible to build earth and rock-fill as well as concrete dams.

We think that the rock-fill dams with thin sloping earth core should be the main type of structures. This type as is shown in chapter I, meets to a greater extent the natural conditions of Iceland as well as local conditions of constructing the majority of dams in the Hvita and Thjorsa river basins. A thin sloping core type demands relatively small volumes of materials that require compaction when being placed.

The slope of a core is recommended in such a way that it might be carried out either after dumping the main volume of rock-fill or simultaneously but stopping the work when weather changes to unfavourable. Nevertheless those unfavourable weather conditions do not prevent rock--fill dumping.

Rock-fill type of structure with reinforced concrete membrane is the next adopted type for dam construction at this area and is recommended wherever the core materials are not available and there is rock in the foundation.

Earth dams out of moraine considered for some projects turned out to be less economical than rock-fill structures that can be explained by the greater volumes, high costs of dense moraine excavation and additional costs of moraine mechanical compaction for those former structures.

The use of loose alluvial deposits the excavation of which is less costly than rock excavation was recommended for such projects where rock excavation out of canals and foundation trenches was not sufficient for construction of rock-fill structures (Hvitarvatn dam).

In some cases when the dam was 10-12 meters high we considered it reasonable from the economic standpoint to recommend concrete gravity structure (Hrauneyjafoss dam). The possibility of installing concrete arch dams was not touched upon since it fell out of the scope of tasks set before us.

It is our opinion that suitable construction materials are present within reasonable haul distance of nearly all the projects. Appraisal estimates can assume, for the most part, the availability and suitability of these natural materials.However, further field reconnaissance with regard thereto must continue. Detailed reconnaissance, sampling and testing are necessary for the definite project planning and design phases of each project.

Standard methods of sampling and laboratory can be found in literature and are not discussed in detail herein.

Location of dam sites and water storages elevations refers to the preliminary Master Plan worked out by the U N experts.

Taking into account that this Report is carried out as a Master-´Plan´ we did not think it reasonable to dwell on numerous problems of general character referring to dam construction since they are extensively covered in the corresponding literature. However, some of them e.g. the dumping of moraine fill under any weather conditions and even at 15° below zero are described in details as it can be applied for the construction of rock-fill dams in Iceland. This method is widely used in the U.S.S.R. and hardly known in the West. Costs and volumes of works have been estimated for dams of all 12 projects on the basis of unit costs given in the AppendixI.These estimated costs would be further exacted after the detailed study of construction conditions of each structure as well as during the construction.

I consider it a pleasant duty to express herein my sincere gratitude to Mr. E.Wessel-Project Manager for his useful advice in my carrying out of this part of our work.

I. NATURAL CONDITIONS OF ICELAND

Geological and climatic conditions are the prevailing factors in the dam construction in Iceland.

From geological point of view Iceland may be considered a young country. It is covered to a considerable part by volcanic products which are mainly basalt lava flows. The age of the basalts differs greatly. The oldest ones belong to the pre-glacial period.

Pre-glacial as well as post-glacial basalt formations consist of several flows divided by contact layers of various sediments of inconsiderable thickness (sandstone and conglomerate) and fragments of volcanic tuff and breccia.

The palagonite formations originating from the period of glaciers movement and created by the volcanic activity occur rather often. They often built up hills and consist partly of tuff, breccia and highly jointed basalt intercalations.

From engineering point of view all these volcanic deposits are of adequate compression strength but their permeability varies greatly and depends mainly on the age of the formations. The older deposits are as a rule less permeable, more indurated and their joints are filled in with secondary, fairly dense materials. The palagonite series which are the oldest formations in some areas in Iceland may serve as an exception here since some of the younger basalts are less permeable than the former. It can be explained by the peculiar conditions of the formation of those deposits that was mentioned above.

The most permeable zones in basalt lavas despite the age of their formation are contacts, contact intercalations between flows and vertical faults created during cooling of lava.

S.Thoroddsen and H.Tomasson's report (1) gives the results of field water pressure tests of lava flows which show a range in permeability from less than 1 LU (Lugeon unit) up to several hundreds of LU but these latter permeabilities, as the authors point out, are found only in young lava flows.

Investigations and explorations of permeability of young basalt lava flows at the proposed Burfell power plant site made it possible

to discern a remarkable peculiar feature of these rocks i.e. the presence of the perched ground-water table. It shows that the vertical permeability of these young basalt flows may be very small.

No less interesting in engineering respect is the fact that river level in Thjorsa and Tungnaa rivers which are flowing mainly on lavas is not connected with the water table. The latter as a rule is found some 10-15 meters below the river bottom. Icelandic rivers carry quite an amount of sediment load consisting of wind eroded volcanic deposits. These materials have so effectively sealed the river bottom that the rivers hardly have any influence on hydrogeological conditions in the lava.

The surface layers of young basalt as it follows from G.Kjartansson's report (5) also have small vertical permeability being sealed to a various degree by fine-grained materials carried by surface waters and winds.

Some places in lowlands of Hvita and Thjorsa river valleys are covered by ground moraine 1-25 meters thick.

Seismic Conditions

Iceland is situated in the Atlantic earthquake region. Earthquakes of tectonic as well as of volcanic origin occur in the country. The former are of a great intensity.

The approximate borders of earthquake regions and values of accelerations are given in the report of a special Commission that studied the problem of earthquakes in Iceland. (2)

The report points to two main earthquake regions. One covers the territory of Hvita-Thjorsa river basins. At the same time it is stressed that the intensity of earthquakes here is the greatest and in some zones the value of acceleration may achieve 0.17 g. For construction of dams and powerhouses in this region the designed value of seismic acceleration was taken at 0,1g. According to 12-ball scale an earthquake corresponding to such an acceleration is considered to be devastating. This probable seismic load should necessarily be taken into account when designing such important constructions as dams, powerhouses etc.

II. GENERAL BEARING OF NATURAL CONDITIONS IN ICELAND ON TYPES AND DESIGNS OF DAMS.

Engineering-geological conditions and comparatively low heights of dams caused by diversion type of the majority of the proposed power plants allow to construct almost at all sites concrete as well as composite rock and earth type dams. The exception would be the sites with the thick glacial moraine layers in the foundation stripping of which would be uneconomically.

Relatively cold winters, great amount of precipitation and strong winds demand durable and frost-resistant materials be used in dam construction and it is especially important for the outer zones of these constructions.

The seismic conditions are the prevailing natural factors on dam construction in Iceland. Later on we'll touch the problem of influence of seismic conditions on rock-fill type dams that according to our opinion would be the main type of dams in construction of hydro power plants on the Hvita-Thjorsa rivers and their tributaries.

In the USSR practice of designing and construction the rock-fill type dams are to meet the following requirements:

Taking into account that dams on rock foundations are less liable to earthquake risk it should be striven that their riverbed section at least be founded on the rock foundation;

The preference should be given to rock-fill dams where leakage-preventing elements are made out of ground;

The great stability against earthquakes is characteristic for dams weighted down vertically by storage water i.e. for membrane-type dams. However the use of ground membranes leads to higher costs of dams up to 10-15% because of gentle upstream slope. Moreover it is rather difficult in this case to secure appropriate junction of the membrane with steep banks and with rear edges of concrete spillway and surfaces of other constructions located in the body of the dam. That is why a compromise solution was adopted and extensively used whereby ground cores are made with a slope in the direction of the upstream reach.

Such rock-fill dams are known as sloping core type dams;

The angle of core's slope may be equal to the angle of natural rock-fill's slope which in this case can be dumped independently of a core and filters. It makes it possible to dump the latter in favourable climatic conditions that is ever more important in view of wet climate in those areas. At the same time the core undergoes lesser deflect with such a priority of construction works because of less rock-fill settlement;

In case of the absence of construction materials for ground core within the economically justified distance dams with reinforced concrete membranes can be built in earthquake regions. In that case membranes should be of a semi-rigid construction or flexible or laminated as they call them for high dams. Moreover there should be unelastic dumping layer of gravel or crushed stone placed under the membrane.

In the U.S.S.R. testing of several rock-fill dam models showed their considerable earthquake resistance. It is explained by the flexibility that makes it possible for the rock-fill to deform without a loss of its stability. The most earthquake resistant are dams built of heterogenous ground and rock materials, the so called composite rock and earth (section) type dams. The ground fills in joints and prevents the occurence of concentrated leakage.

The specific construction features of rock-fill dams designed in conditions of seismic regions which were taken into account by us when working out dam designs for hydro power stations on the Hvita-Thjorsa rivers are the following:

Freeboard is chosen in view of possible waves resulting from seismic shock. At the already built dams the freeboard ranges from 1.5 up to 3.0 meters, settlement allowance not included. We have taken the latter figure which should be defined more accurately later after calculating the possible height of waves against upstream slopes. Width of a dam's crest is taken greater in seismic regions than elsewhere.

The minimum crest width is usually taken as 0.1 of the dam's height but never less than 5.0 meters. In view of relatively small heights of the dams we considered it possible to take the crest width equal up to 6.0 meters.

The stability of dams in earthquake areas deem it necessary to make sloppes more flat. According to Soviet standards slope angles should be increased by 10-20 o/o when carrying out preliminary design work for regions with design seismic acceleration of 0.05-0.10g. In designing and estimating volumes of dams we have thus increased slope angles by approximately 10 o/o.

In more detailed dam design such as calculating stability of slopes the design acceleration at the top of a dam should be taken several times more than at the foundation depending on the dam's height. The difference in acceleration values at various heights of a dam was recorded by seismographs installed in some dams in the U.S.S.R. as well as in other countries.

To prevent piping in the impervious core in case of joints and cracks being formed by earthquakes the thickness of protective filters is increased by 2-3 times. In non-seismic regions the volume of core protecting filters in dams is usually 6-8 o/o. We estimated it sufficient to take this volume at 12.5 o/o.

This value must necessarily be further adjusted taken into account qualities of materials used for protective filters and transition zones. Usually materials for core filters for dams in seismic regions should be of higher quality than elsewhere.

Thus we conclude that the most suitable type of dams for southwestern Iceland would be rock-fill dams with slope or vertical core or composite earth and rock section type dams constructed in accordance with requirements resulting from natural conditions mentioned above.

However this does not exclude the construction of other types of dams out of natural construction materials and even concrete ones which in some conditions can turn up to be more economical as will be stated below.

III. HVITARVATN DAM

Hvitarvatn dam is the first project at the upper part of the Hvita River originating from Lake Hvitarvatn. From its very effluent from the lake downstream to Aboti waterfall beginning about three kilometers downstream from the road bridge across Hvita, the river flows on almost a flat grade. At the section between the bridge and waterfall the Hvita River takes in its tributary Jokulfall with the drainage area almost equal to the half of that of the Hvita River.

The River valley in this area is characterized by relatively plain relief. There are only Lambafell and Blafell mountains 100-150 meters high at the right bank where Aboti waterfall's rapids begin. The northern slopes of these mountains could probably have been the Lake's bank. Spurs of a small hill come close to the River here. That is why this section is considered suitable for location of the dam that will have a relatively small length and will provide for the diversion of Jokulfall into the Lake.

The bedrock of this area is composed of two basalt formations of different age and permeability and also of breccia formations being the oldest ones and these are the main rocks of Blafell and Lambafell mountaines. The Hvita River flows on the old dense basalt consisting of several flows with the thickness of some tens of meters. The surface of this basalt is covered with ground moraine of 10-15 meters thick. This moraine appears to be from Glacial Period, it is very dense and probably impermeable.

Upstream from confluence of Jokulfall with Hvita grey Basalt outcrops cover a considerable area of the bottom of the proposed reservoir. This basalt is younger and with numerous joints unfilled with secondary materials. It is covered with a layer of dense moraine 20-25 meters thick. Basalt however lies exposed on the bank (slopes) of the River. If the dam site is located between the confluence of Jokulfall with Hvita and Aboti waterfall the dam's foundation would be on the old basalt in riverbed section and on moraine in the floodland section. The section of the dam between slopes of Blafell and Lambafell mountains having the height of 10-12 meters will be the only one founded on the Grey basalt with the thickness of up to 40 meters.

The breccia formations composed mainly of volcanic tuff with basalt pillow lava intercalations will be in the foundation of the dam's right abutment along the length of 100 meters.

Another possible location of the dam is the Hvita River section some 300-500 meters downstream from the bridge. The dam would then have a considerable length to provide for the diversion of Jokulfall into the Hvitarvatn Lake which would serve as a storage reservoir and its level should be raised by 20 meters above the present one. This site is certainly less favourable from the geologic point of view than the site discussed above since Grey basalt will be in the dam's foundation here. However, a dam will be lower here because of the higher bank elevations but the greater distance from the waterfall would necessiate the prolongation of the headrace tunnel and it will also lead to a certain head loss. The comparison of the additional tunnel costs with the possible gain in the dam showed that the two are almost the same. From the geologic standpoint the lower site is preferable. Many watersprings were observed in the moraine during field investigations of the moraine of the upper site which proves to the high permeability of the moraine in this area.

The construction materials are available at both sites and consequently it is our opinion that the site closer to the waterfall is preferable.

The water levels of the river at the mentioned possible sites have elevations 418 and 415 meters respectively. The dam should create storage with the elevation of 440 meters that will require raising of the present Lake level by 20-25 meters. Thus the maximum dam's height might be approximately 28 meters. The foundations at both sites would be in ground moraine 10-15 meters thick. In view of the small height of the dam especially in the floodland sections the moraine should be left as it is since its excavation can neither be stipulated by technical requirements nor justified economically. A concrete dam is unacceptable here despite the fact that in all reports on the geology of the area among which Kjartansson's (3) is the most exhaustive one glacial moraine is described as strongly indurated. Only rock-fill or earth constructions can be built here. The type of dam should be selected taking into account qualities and quantities of available materials that should be excavated when constructing other structures for the project.

Dam construction practice proved convincingly enough that economic solution can be achieved only with full use of these materials.

Thus we have considered two dam alternatives i.e. rock-fill with thin sloping core and earth section type which is earth and rock-fill dam with wide moraine core. The latter alternative was considered because the amount of rock excavated from the tunnel is insufficient for the rock-fill dam while there is ample amount of moraine at the site.

The dam with the top's width of 6.0 meters and the total length of 3050 meters (consisting of two parts) has the maximum height 28 meters in the riverbed section. The upstream slope 1:2 and downstream slope 1:1.6. The location of the dam corresponds to the lower of the two sites discussed above.

The rock and earth section dam had the same top's width, length and height. Upstream and downstream slopes are 1:2.5.

The profile was drawn up on the basis of topography on a scale of 1:10,000 with five-meter contours.

We have estimated the volumes of the main works as the following:

1 The rock-fill dam

a/rock-fill-	1,060,000	cu.m.
b/core-	187,000	cu.m.
c/filters-	301,000	cu.m.

2 The rock and earth section dam

a/rock-fill-	744,000	cu.m.
b/core-	854,000	cu.m.
c/filters-	102,000	cu.m.

The comparison of cost estimates of these two alternatives showed that rock and earth section dam is by some 10 o/o more expensive than the rock-fill dam (the adopted prices will be dealt with below). The rock and earth section dam besides being more expensive has in this particular case one more serious disadvantage caused by specific construction conditions in Iceland. The placeability of moraine with heterogenous structure is greatly dependant on its moisture content. The range of optimum moisture content for compaction of moraine is very narrow. It largely explains the difficulty of moraine compaction in areas with heavy precipitation and frequent strong winds which is the case here in this area.

That is why it is quite natural to strive to minimalize volumes of moraine fills. All the above-said favours the selection of the rock-fill dam with thin sloping core alternative which we propose as the main type for the Hvitarvatn project.

The recommended site is at natural river level of 415.5 (cf the map on a 10,000 scale). The dam as has already been mentioned consists of two parts divided by Lambafell mountain. The total crest length of the dam is 3050 meters. The maximum height in the riverbed section is 30 meters. The riverbed section of the dam is the rock-fill structure with the sloping core made of moraine.

For the right bank dam divided into two parts by the spillway for the better junction with abutments and in view of its small height the rock-fill type with the vertical core is recommended.

The thickness of a core which might be made of moraine is defined by the design pressure gradient equal to 5. The core abuts to bank moraine as well as to the rock in the riverbed by means of the cut-off. The lower part of the fill is seperated along the contact with moraine foundation by horizontal inverted filter.

The moraine is impermeable enough so that no anti-leakage measures besides the cut-off will be required at the bank sections where the head does not exceed 18 meters. The cut-off should be taken down to the basalt's roof in the places where moraine layer is not more than 5-6 meters. It should penetrate the rock to not less than 0.5 meter.

The basalt flooring the riverbed section was not tested on the water absorption. We consider it necessary to grout a curtain here taking into account that the head is 30 meters. For cost estimates we provisionally accepted a single-row curtain with 3-meter space between bore holes using 200 kg. of cement per running meter of a borehole. The curtain should reach down the depth where water absorption won't be more 0.05-0.10 l/sec. Lugeon criterium applied formerly for up to 30 meters high dams equal to 0.03 l/min led to the unjustified increasing of grout curtains costs. The experience in the U.S.S.R. and other countries made it possible to apply new less strict requirements and to differentiate between curtains in accordance with the head.

The permeability of the grouting curtain in this case should not exceed 0.03-0.05 l/min. The spillway is a low concrete sill equipped with fish-belly gates. The installation of the gates is stipulated by the possibility of wind-driven large pieces of ice broken loose of Langjökull glacier.

The spillway is designed to pass a flood discharge up to 700 m^3/sec and consists of 6 bays by 15 meters wide with 3-meter depth at the sill.

The location of the spillway is chosen in such a way that the bed of the brook there may serve as a tailrace canal.

The diversion of the river in the construction period for the foundation treatment of the riverbed section and for the construction of the dam's lower part might be carried out through the tunnel using for this purpose the upstream section of the power house's permanent tunnel. The diversion will require an upstream cofferdam that might be included in the body of the rock-fill dam.

The rock excavated from the tunnel will be the main source of fill material. However the amount of this material will meet only 15 o/o of requirements in rock for the dam construction. The rest amounting to some 850,000 cu.m. should be excavated from the quarry. There should be two of such quarries located on each bank of the river. The slopes of Lambafell mountain at the right bank composed of palogonite rocks will be a very convenient place for a quarry. The mentioned rocks are suitable for rock-fill at least in the inner zones of the dam. The excavation of the quarry rock might not be the most economical solution of the problem.

At the adjacent upstream section the Hvita River has a small hydraulic gradient. The riverbed and the banks are most probably alluvial deposits of gravel and sand. These materials can be excavated for approximately half the excavation costs of the quarry rock. The alluviums can be easily used for filters as well as for the making the central part of the dam, behind the sloping core. We can point out Holjes dam in Sweden as an example of such a solution.

In designing the dam it might be advisable to consider an alternative with moraine core the thickness of which would be intermediate between the two adopted for the above-discussed alternatives.

This alternative is stipulated by the wish to fully use the economically justified amount of moraine the excavation of which would be slightly less costly and the hauling distances definitely shorter.

After having estimated volumes of work for Hvitarvatn dam we estimated dam costs in accordance with the recommended type of the dam with spillway and foundation treatment costs included i.e. costs of all features related to the dam.

Estimated cost of the dam is 5.4 million dollars. This sum might be diminished on account of the use of cheaper materials for the inner zone of the dam that was already mentioned.

Estimated cost of this dam as well as of all other projects is based on unit prices for main types of works given in the Appendix.

Unit costs are based on Icelandic price list of 1965. However some uncertainty exists and the cost estimates should be regarded as preliminary ones. Taxes and duties are not included.

We find it possible to take 3.2$ as a unit price per cubic meter of moraine fill.

Unit price of excavation of rock from quarries and broad canals equals 4$ per cubic meter in natural conditions. In estimating cost of rock-fill we have taken into account the loosening up to 30 o/o. Thus one cubic meter of rock-fill was estimated at 3.1$.

We supposed that some 50 o/o of the inverted filter materials would be a natural sand-gravel mixture and the rest-artificial mixture. That is why the cost taken for filters at 4.5$ is the intermediate between the cost of the natural and artificial materials.

The concrete structures were estimated using two costs the difference being dependent on the reinforcement content with costs of metal used for fixing of forms included. The forms costs were related to that of one cubic meter of concrete masonry and are estimated proceeding from the average volume of a concrete block being equal to 500 cu.m.

When the reinforcement content is 15 kg. in one cubic meter of concrete the cost is 29.4$ and when this content is 40 kg. the cost is 36.5$.

The former cost was applied for the concrete of spillways and the latter to reinforced concrete membranes of dams.

The cost of gates and their hoisting gears was estimated by increasing the cost of one ton of gates metal by 30 o/o.

IV. GULLFOSS DAM

The Hvita river flows in a narrow gorge for 400-500 meters in the proposed location of the Gullfoss power plant. The upstream section of the river where rising ground of the right bank extends close to the water edge seems to be the most favourable for the dam location from the topographic standpoint. The left bank topography is rather uniform with the over flood land surface direction parallel to the riverbed. The water level elevation is 213 meters. The average discharge of the river here is 120 cu.m/sec.

In engineering-geological relation this site has considerable advantages as the old basalt floors the riverbed and outcrops in bank slopes. However the left bank is covered with moraine several meters thick. Moraine deposits were observed in the right bank as well.

Basalt of the foundation is apparently represented also by two or more flows the proof of it being the presence of sedimentary rocks of conglomerate and sandstone some 10 meters thick. This layer can be traced in both banks but at different levels. Mr.Nicol supposes in his Report (3) that a fault might lie in the right bank. We could not find any more definite data on this phenomenon. It should be clarified by further more detailed investigations. It would be necessary to find out the probability of the further extension of the fault as well as quality and permeability of the materials filling its joints. If the findings would prove that such development does not take place the dam can then be built here. But grouting difficulties will remain since breccias usually filling this joints are rather difficult materials for grouting. The Boulder dam in the U.S.A. may be referred to as an example here.

The sandstone of the mentioned intercalation is fine-grained, loose and easily eroded. When the reservoir would be full this intercalation may serve as a path of the intensive leakage accompanied by washing out the sandstone.

Mr. Nicol describes basalt formations as having many joints. But since these joints are filled with clay materials the formations are of low permeability.

The dam height in the riverbed section will be 37 meters with water storage level at 244 meters. The average dam height at the left bank where it has the greatest length won't exceed 25 meters.

To select the type of a dam we proceed from the following considerations. The possibility to use the rock excavated from tunnels, canals, foundation trench of the spillway and the head and tailrace canals. It would be reasonable to use these materials in the fill dam. The surface moraine is several meters thick (unfortunately more detailed data are not available). The height of the dam being 25 meters the excavation of the moraine would be unadvisable since it'll lead to the so far unknown increase of the dam's height on account of the most voluminous downstream section of the dam. Therefore the moraine but the upper vegetation layer should be left in the dam's foundation. It might be expected that moraine here would be found suitable by its granulometric composition for the core.

In view of the above-said we came to the conclusion that the rock-fill construction with the core would be the most preferable. The cut-off which is the continuation of the core might be taken down to the rock if moraine in natural condition would turn out to be highly permeable as it is the case near the bridge across the Hvita river not far from the Hvitarvatn lake.

The core may be either sloping or vertical in case the spillway divides the dam.

In designing the dam we placed the spillway on the right bank where elevations range from 235 up to 237 meters.

The dam has a vertical core seperated from the fill by filters that are also provided for at the contact of the downstream section of the fill with the moraine foundation.

We do not consider the location of the spillway selected by us to be the most favourable one. It is desirable to choose another location of the spillway when topographic conditions at the dam site would be defined more precisely.

The discharges at this section being relatively small the river can be diverted during the construction period through the tunnel feeding the power plant.

It will enable to avoid the using of the incompletely built spillway for the construction discharge and therefore the spillway might be located outside the dam at the higher elevations. Such a location of the spillway is desirable because there won't be any necessity then in junction of the rock-fill dam's core with rigid concrete abutments of the spillway. In this case the core selected might be a vertical one.

The total dam's length along the crest is 475 meters, the crest width is 6.0 meters at the elevation 247.0 meters. The plan and cross section of the dam are given in the Appendix. The value of settlements at the crest is included in the freeboard. The crest level may be slightly lowered here since the height of the bank sections of the dam is considerably less than that of the riverbed section (some 12-15 meters).

It is utterly important to grout a curtain to prevent the possibility of high leakage and washing out of sandstone out of sedimentary intercalation. The curtain might be placed under the dam's cut-off. The grouting section would be defined after more detailed investigations of the area with the purpose of determining whether the porosity of the rock allows such measures to be carried out or not. If the investigation data would confirm this possibility the height of the curtain might be limited by the thickness of the intercalation with a slight penetration of the former into the basalt as to guarantee that the whole thickness of the intermittent layer is sufficiently watertight.

In case the rock would turn out unsuitable for grouting mixture injections an alternative solution might be adopted i.e. the banks surface within the limits of the sedimentary intermittent layer is covered with a layer of low permeable materials. The length of such a shield cover might be calculated on the basis of the allowable pressure gradients.

The gradient is usually found according to the data of laboratory tests on piping. The obtained limit gradient is divided by two-three times and then adopted as the design one.

However to carry out this cover would necessiate the flattening of the banks at a certain distance upstream from the dam as they

are very steep in natural conditions so that the dumping and compaction of the ground would be difficult to manage.

The spillway is designed to pass high flow discharge up to 2100 cu. m./sec. Since as in case of the adopted by us spillway location the volume of work for the rock-fill dam depends on its length consideration was given to several spillway alternatives with difference in height of gates and consequently in sills elevation. Moreover the different spillway's length requires different quantities of rock excavations from head and tail race canals. We have considered spillways with the height of gates 3.5 and 7 meters. After volumes of work for the rock-fill dam as well as for the spillway have been calculated including all the accompanying expenditures it was concluded that the most economically advantageous is the spillway alternative with the gates height of 5 meters. In this case the dam's length will be 100 meters and it consists of 6 spans by 15 meters closed by tainter gates. From the design standpoint the spillway is a low threshold divided by piers 11.5 meters high and 1.8 meters thick.

The volumes of work are estimated as the following:

	Excavation of soft ground-	10,000 cu.m.
	Excavation of rock-	7,100 cu.m.
Fill.	Rock-fill-	201,000 cu.m.
	Core-	43,000 cu.m.
	Filters-	61,000 cu.m.
	Concrete and reinforced concrete-	6,000 cu.m.
	Grout curtain-	1.2 ton of cement
	Gates and hoisting gears-	164 tons

Estimated dam cost in accordance with unit costs given in the Appendix 1 is 1.71 mil.$.

It should be noted down here that costs of the dam alternative with the spillway taken outside of the dam were estimated as practically the same.

V. NORDLINGAALDA DAM

The dam will be located in the upstream section of the Thjorsa river and serve as a storage dam for all power plants on the Thjorsa river.

The Report of Icelandic geologists Messrs. T.Tryggvason and T. Einarsson (6) gives the description of the two possible dam sites from the standpoint of geology and topography.

The upper site is near Soleyarhöfdi and the lower near Nordlingaalda. As can be deduced from the Report the upper site is certainly less favourable than the lower one with regard to the topography and the foundation geology. With the dam located at the downstream site the storage volume might be larger (all other conditions being the same). Taking into account all the above-said we found it possible to give preference to the site near Nordlingaalda. However it is reasonable to locate the dam in the riverbed section som 400 meters upstream from the confluence of the Svarta with Thjorsa where the Svarta flows parallel to the riverbed for the purposes of the more convenient diversion of the river during the construction period. This location as compared with the one proposed by the authors of the Report would allow to decrease dam's height in the riverbed section by several meters. The river level at this site as can be seen on the map on a scale 1:20,000 is equal to approximately 557 meters.

Unlike the above-described sites the foundation here is mainly in sedimentary rocks. The riverbed cut down by the river is floored with conglomerate, the banks with thick layer of the dense moraine. The thickness of the moraine layer as can be seen in the cross section in Tryggvason and Einarsson's Report (6) is some 30 meters. Moraine is under a layer of volcanic breccia 3-6 meters thick. This breccia seems to be highly permeable as water springs are observed at the exposure of this rock in the bank slope. The conglomerate underlying the breccia interlayer is tight and firm according to mr. Nicol's Report (3).

Nordlingaalda storage was estimated and on its basis the water storage elevation taken at 593 meters. Thus the maximum dam height will be about 45 meters and not more than 10-15 meters along the greater part of its length. The dam is so designed as consisting of several sections divided by the hills in the area. The total dam's length (or the total length of all its parts) is 7.2 km. This length was defined for the adopted dam crest elevation 597 meters i.e. 4 meters higher than the natural water storage elevation. We deem it necessary in view of the strong winds prevailing in this area and the possibility of wave overbreak in case the elevation difference would have been 3 meters.

No other constructions but the spillway are envisaged since building of power plants here is thought to be unreasonable. Thus no borrow pit materials would in fact be available. All the materials for the dam should be excavated from quarries. There is a possibility of a quarry for rock excavation from the basalt to the south from Eyvafenskvisl on the right bank as well as recent alluvial deposits on both banks not to say of moraine available in the vicinity.

Thus we came to a conclusion that the dam at this site might be an earth one made of moraine (i.e. rock and earth section) or a rock-fill construction with the greatest possible use of alluvial deposits for the dam's central core fill located beside the sloping core. Moraine deposits with the required content of fine particles might be discovered here to be used for the dam's core. A certain part of the rock for the dam might be substituted for the alluvial materials that will be economically reasonable since the excavation of the latter will be 1.5-2.0 times cheaper than that of the rock.

Because of the scarce reconnaissance data on the deposits at the dam site we've decided to consider the two dam alternatives i.e. earth out of moraine and rock-fill with a sloping core out of moraine.

When estimating volumes of work for the two alternatives we used the topographical map on a scale of 1:20,000.

The surface weathered moraine layer the approximate thickness of which we estimated in our calculations as one meter should be stripped off before constructing the earth dam.

Such a junction with foundation on account of the heads that will be at Nordlingaalda dam make any grout curtain unnecessary except may be in the riverbed section. As this section is very short in comparison with the total dam's length the grout curtain costs can not considerably influence the total dam costs. That is why we have not included any grout curtainsin our cost estimation for the earth dam alternative. It does not however in any way exclude that some grouting in the breccia layer might be necessary.

Another case is with the rock-fill dam alternative. To lengthen the leakage path either the deep cut-off being the vertical continuance of the sloping core should be installed here or a shallow grout curtain that should be taken down into the conglomerate by 0.5-1.0 meter.

The stripping off the surface moraine layer before placing the rock-fill is not required in the case of a construction. Grouting curtain cost was included in the estimated costs for the rock-fill alternative. Such curtains can be successfully constructed in moraine under certain sonditions e.g. at Mattmark dam in Switzerland this was done.

Calculated volumes of work for the main dam alternatives are the following:

Earth dam alternative

-ground excavation-	1,000,000 cu.m.
-moraine fill-	4,100,000 cu.m.
-filters-	225,000 cu.m.
-rock-fill	884,000 cu.m.

Estimated earth dam alternative cost- 19.4 mil.$

Rock-fill dam alternative

-ground excavation-	156,000 cu.m.
-rock-fill-	2,248,000 cu.m.
-moraine fill-	524,000 cu.m.
-filters-	665,000 cu.m.
-grouting-	3,640 tons of cement

Estimated rock-fill dam alternative-cost- 12.3mil.$

Estimated cost comparison shows absolute advantage of the rock-fill dam alternative. Thus we recommend for this project the rock-fill dam construction. In case further reconnaissance finds alluvial deposits sufficient for the fill of a dam and within economically justified distance the earth with moraine core dam constructed of these materials might turn out to be cheaper than the rock-fill dam construction.

The spillway is calculated to pass a flood discharge up to 2000 cu.m/sec. and has the length of 100 meters and is located at the extreme right bank section of the dam. Its location so chosen as to divide the spillway at the small height section of the dam and to use the bed of the stream for the diversion of the spillway's water.

VI. HVANNGILJAFOSS DAM

The project will be located at the Thjorsa river section between the Nordlingaalda dam and the Dynkur power plant. The site was chosen in the 600 meters distance from the waterfall where the river water level elevation is 506 meters. The dam here will cross the Thjorsa riverbed as well as two other streams flowing into Thjorsa in a short distance downstream from the fall. We deemed it necessary to consider Hvanngiljafoss dam at the three alternative water storage elevations 520,545 and 570 in view of the relatively high cost of the Nordlingaalda dam and of the possibility that the recently stated volume of the Nordlingaalda water storage might be less than was shown by the calculations made on IBM computer. After the costs analysis the water storage elevation of 525 meters was chosen.

As is stated in Mr. Nicol's Reports on the geology of the area (sections are not available) the river flows here on the well cemented conglomerate overlain on the banks by the thick ground moraine layer. The maximum dam height here is 23 meters. The rock-fill or earth (of moraine) constructions are feasible here on account of the available materials. However as was previously shown the moraine dam would be more costly than the rock-fill dam. Estimated rock excavations from foundation trench and spillway canal seem sufficient for the rock-fill construction. Thus the rock-fill construction with sloping core will be certainly the most suitable as the construction with reinforced concrete membrane is unacceptable because of the thick moraine layer in the foundation of the dam.

The crest elevation is 528 meters, the crest width-6 meters, the dam's length along the crest-1600 meters. The upstream slope 1:2,, the downstream-1:1.5. Unfortunately there is a lack of reconnaissance data for this site so nothing can be said about the availability of filter materials and of the suitability of the moraine as core material.

The longer part of the dam will be located on the right bank and the dam's height of this section will be 12 meters. The carried down

core's cut off would be sufficient for the junction of the core with the moraine foundation.

The riverbed section of the dam is founded on the well cemented conglomerate usually containing lenses of loose sandstone. It necessitates the grouting of a curtain at this section which is accepted by us as a single-row curtain with the length of 500 meters, 10 meters deep with boreholes at a 3 meter distance. Undoubtedly these figures might be changed after further investigation of the foundation (but it is certain that the grout curtain is necessary).

The spillway is located outside the dam on the left bank and has 6 spans by 15 meters with low sills. The tainter gates were chosen. The total length of the head and tailrace canals is about 600 meters. The chute is planned beside the downstream canal.

Power house's intake is envisaged at the dam. The penstocks should then be placed along the downstream slope of the dam.

To lessen the required volumes of ground excavations it is convenient to locate the power house building in the bed of the stream.

The estimated volumes of work for the dam with the spillway and the canals are the following:

-moraine excavation-	104,000	cu.m.
-rock excavation-	600,000	cu.m.
-rock-fill-	566,000	cu.m.
-core-	105,000	cu.m.
-filters-	160,000	cu.m.
-concrete-	4,300	cu.m.
-grout curtain-	250	tons of cement
-gates and hoisting gears	126	tons.

The volume of moraine excavation equals the volume of the vegetation and weathered layer one meter thick.

The estimated dam costs-2.4 mil.$.

VII. DYNKUR DAM:

The site of the dam is located between two waterfalls where the river elevation is 453 meters. The bank slopes at the site are relatively gentle despite the closeness of the waterfalls. The main rock of the site is old basalt (as stated in Mr. Nicol's Report (3)).

In the strambed the river seems to wash out basalt down to sedimentary layer lying between basalt flows and composed of sandstone and conglomerate.

Neither of these rocks was investigated on the permeability. It might only be supposed on account of the age of basalt that its numerous joints are filled with sedimentary materials. Conglomerate is also undoubtedly suitable and impermeable. Sandstone however is loose and might be eroded after the dam is constructed. Data on the deposits of other construction materials are limited to the stating of their availability within a reasonable distance. Nothing is said about their characteristics.

The Dynkur power plant is of the diversion type with a long headrace tunnel which longer part is passing through good basalt. This rock can be used for the construction of the dam. The dam's height is chosen at 43 meters with the water storage elevation 490 meters and crest's elevation 493 meters.

As the dam's height is 25-30 meters in average and the excavated rock is available a rock-fill construction would be reasonable at this site, but unfortunately data on other construction materials are insufficient so a definite recommendation of the rock-fill construction is impossible. Nevertheless it seems advisable to build a rock fill construction here with a core that to a greater degree meets the Icelandic requirements than a construction with the reinforced concrete membrane.

Proceeding from the above-said and on the basis of topography on a scale 1:20,000 the dam's length along the crest is equal to 1100 meters. The crest width is 6 meters. The upstream slope is 1:2,4, the downstream-1:1,6. In view of the small average dam's height it might not be curvilinear in plan without a camber upstream.

However it would be necessary at the left bank abutment of the dam where this camber is needed for the junction with the high part of the bank.

After the construction of the diversion canal there would be two river branches upstream from the dam that will allow to shift the flow from one to another during construction period for the foundation treatment and dumping of the lower section of the dam.

When the river is completely dammed (i.e. both branches) river flow can be passed through a tunnel, the power plant headrace tunnel serves as a part. Cofferdams are included in the body of the dam and in estimated volumes of work were not considered separately.

The spillway is calculated to pass a flow up to 2500 cu.m./sec. and is composed of 7 spans 15 meters wide. The tainter gates are 5 meters high. The spillway is located on the left bank outside the dam.

Anti-leakage treatment of the dam's foundation would be different as was the case for all above-considered projects. The most difficult would be to protect the riverbed section because of the layer of conglomerate and loose sandstone present there. This layer is some 15 meters thick and it would be difficult and costly to cut into it with a cut-off. The grout curtain construction would be definitely much cheaper. The edge lenght of the curtain should be defined after investigating the piping resistance of the interbed material. The total grout curtain length for our preliminary estimates was taken equal to 400 meters with the distance between boreholes-3.0 meters.

The carrying down of the core's cut-off for the rest part of the dam should be sufficient as basalt on the banks has low permeability. The depth of the cut-off might be defined by providing the pressure gradient not less than 6.

Before selecting the water storage elevation we estimated volumes and costs of works for the three water storage elevations i.e. 480, 490 and 520 meters. The cost of one kilowatt of installed thermal plant capacity equal to 300$ served as the criterium of the economic reasonability of water storage elevation raising. The estimates showed the optimum water storage elevation to be within the limits of 490-492 meters.

The estimated volumes and costs for the dam at the water storage elevation 490 meters are the following:

-rock excavation-	78,000 cu.m.
-rock-fill-	866,000 cu.m.
-core-	170,000 cu.m.
-filters-	172,000 cu.m.
-grout curtain-	789 tons of cement

The dam's cost- 4.3 mil.$

The water storage volume- 340 mil.cu.m.

VIII. SULTARTANGI DAM

This project will be located upstream from the Burfell power plant near the confluence of the Tungnaa river with the Thjorsa river. The left bank at this section is very flat and the fall of the Tungnaa river flowing here is small so that it is divided into several branches. There is a waterfall at the point of confluence of the Tungnaa river into Thjorsa. The characteristic feature of the dam's foundation is interbedded layers of lava flows and loose sand. The steep right bank rests on conglomerate.

Downstream from the waterfall Thjorsa flows on firm and tight basalt. This rock is covered on the left bank with thick layer of lava and only at some elevations outcropping basalt hills can be observed.

The water storage elevation was chosen at 300 meters.

The consideration was given to the two economically reasonable dam sites. The lower being located downstream from the waterfall and the upper dam site in three kilometers upstream from the waterfall. The lower dam site seemed more favourable since the dam here could have been considerably smaller and the geology of the right abutment better than at the upper site.

Nevertheless the estimated volumes of materials showed that the dam at the upper site would be more economic. Thus it was adopted for the further design work.

At the water storage elevation 303 meters the dam's length along the top would be 5600 meters, the maximum height-17 meters. The ravine on the left bank filled with sand was thought advisable to use for the power plant diversion canal the building of which in this case would be located at the dam's left abutment. The relatively shallow water storage made it necessary to suppose a headrace canal 3000 meters long. The canal will be in lava. The volume of excavation from the canal equals approximately one million cubic meters. Thus a rock-fill construction is more preferable. The estimated volumes of rock-fill showed that the volume of rock excavated from the headrace canal and the power plant's building bottom trench would be sufficient for this purpose. The ample amount of available moraine gives rise to a hope that deposits of materials suitable for the core might be found here.

The spillway is composed of 12 spans by 15 meters with the gates 5 meters high. The number of spans was defined by the accepted high flow discharge equal to 4300 cu.m./sec. The spillway would be reasonable to locate in the Thjorsa riverbed on sound basalt. During construction period the Thjorsa river might be diverted to the Tungnaa river through the special connecting canal. After the piers are built and the Tungnaa river is dammed both rivers can be passed through the spillway without sills. In view of the required junction of the dam with the concrete of the spillway it is desirable the core to be a vertical one.

Mr. Kjartansson supposes in his Report (5) that lava at this area might be of relatively low permeability but probably except the upper zone with many joints.

There are no available data on the permeability of the lava (as it is the case for all other projects). Consequently taking into account the relatively small dam's height and the above-stated supposition we found it possible to recommend here a trench cut-off without any grout curtain. The vegetation layer should be stripped off only within the area of rock-fill dumping. Moraine and upper lava layer under the rock-fill might be left. The depth of the trench cut-off would be defined by the most economical combination of foundation leakage and construction costs.

Conglomerate on the left bank will require the most detailed exploitation. At the present stage of knowledge of the rock's qualities it might be recommended to construct here a trench cut-off to basalt's roof penetrating 0.5-1.0 meters into the rock. The permeability of contact zone between lava and basalt on the left bank of Thjorsa is not known. This contact surface outcrops on to the bankslope. When the storage would be filled short leakage paths might appear along this plane. Preliminary to anti-leakage measures detailed investigation of this zone is necessary.

Volumes of main works for Sultartangi dam are the following:

-rock-fill-	1,052,000 cu.m.
-core-	185,000 cu.m.
-filters-	180,000 cu.m.
-concrete-	32,500 cu.m.
-gates and hoisting gears	252 tons

Cost of dam equals 6.40 mil.$.

IX. SKARD DAM

The dam would be the next power plant after Burfell on the Thjorsa river. The Skard water storage would be the lower reach of the Burfell power plant and its water surface elevation would be 120 meters. The dam site is located in an area where the river has elevation 107 meters. The topography and geology of the area is but slightly studied. It is stated in Mr. Nicol's Report that the main bedrock in the riverbed on the left bank is post-glacial Thjorsa lava. However in the geologic cross-section accompanying the Report it is shown that the river flows on old basalt and lava can be observed only close to the left bank. The thickness of the lava ranges between 12-15 meters. The intercalation of post-glacial sediments containing gravel, sand, silt and probably clay is shown between lava layer and basalt's roof. There are no data on the permeability of this intercalation. In the table attached to the above-mentioned Report these fini-glacial sediments are given general characteristics from which can be deduced that at the Thjorsa river basin such deposits consist of loose mixtures of sand and well-rolled gravel. Nothing is known about other materials in the vicinity of the site which might be used for the construction of various structures. The discharge at this section of the river exceeds 300 cu.m./sec. and therefore it won't be feasible to use the tunnel for the river diversion during construction period. Consequently the spillway's bottom trench should be so deep as to allow the whole river flow to pass through the spillway without sills when the riverbed would be dammed.

Thus borrow-pit materials would be excavated from the power plant's tunnel, canals and the spillway's bottom trench. All these excavations would be carried out in old basalt. The excavated rock might be used for the dam as well as aggregate material for the spillway's concrete.

The dam's height with the three-meter freeboard would be 22-23 meters.

We think the rock-fill construction would be the most economic solution for this site in view that the volume of borrow-pit materials is sufficient for the mentioned type of construction. To secure the more safe junction with the spillway's abutments a dam with a vertical core is desirable here so that's what we have adopted for estimating the dam cost.

The dam's length along the top is 830 meters, the top elevation 123 meters. The upstream slope is 1:1.6, the downstream 1:1.5.

In order to lessen the leakage from the storage and to prevent washing out of fine-grained materials from sedimentary intercalation we find it necessary to envisage at the present stage of knowledge to envisage a grout curtain though the head at the dam is rather small.

Certain amount of time will be required for the dam's foundation to get sealed after the storage is full. Most damages at dams because of the washing out of foundation sedimentary rocks occur during first years of their operation.

The required grout curtain with the total length of 500 meters consists of the two sections one being in the area of lava layers the other in basalt. The main difference between the two sections is in the cement expenditure which was adopted 280 for lava and 150 kilogramms per running meter of borehole for basalt.

The spillway is designed to pass 4300 cu.m./sec. and is composed of 6 spans of 15 meters. Tainter gates are 6 meters high.

The bottom openings 3x3 meters two in each spillway's sill are envisaged to pass a part of the design flow and to diminish the amount of rock excavation from canals and the bottom trench. For the reason stated above the bottom of the spillway's canal is at the riverbed level.

The estimated volumes of the main works for the dam on the basis of a 1:50,000 scale map are the following:

-excavation of soft ground- (vegetation layer one meter deep)-	11,000 cu.m.
-excavation of rock-	142,000 cu.m.
-rock-fill	232,000 cu.m.
-core-	41,000 cu.m.
-filters-	43,000 cu.m.
-concrete-	27,300 cu.m.
-grout curtain-	370 tons of cement
-gates and hoisting gears-	168 tons

The total estimated dam cost with the spillway, canals and cofferdams included equals 2.26 mil$.

X. URRIDAFOSS DAM.

This dam represents the last stage of the power plants in the Thjorsa river. Possible dam location is discussed in Mr. Thoroddsen's (4) and Mr. Kjartansson's (5) Reports. They proposed four dam sites each one of which is acceptable but has its advantages as well as disadvantages. We had no possibility to consider all the sites and confined ourselves to one which we selected as the most preferable from the standpoint of the topography and geology. At the selected site the ground base is of tight and firm pleistocene basalt cropping out of lava hill on the right bank which is called Sandholt. This basalt layer floors the riverbed section on the left bank. In case the dam is located at this site the tunnel would be somewhat shorter than at other sites except no 4 but there the dam would require greater amount of materials in comparison with other dam sites, which is caused by low riverbed elevation and little relief gradient of the right bank at the site 4 that would require an extension of the structure.

The maximum possible Urridafoss water storage elevation is 43 meters. The dam's foundation along its whole length is in old basalt.

The choice of construction type for this dam site was governed to a greater degree by water discharges that should pass the spillway than by the availability of construction materials at site. At the adopted by us design flow being 4500 cu.m./sec. in view of certain discharge transformations on account of the water storage the required spillway's length would be 130 meters. In defining this length we have taken rather considerable tainter gates height equal to 7 meters in order to avoid voluminous rock excavations on the river banks. Water discharges during construction period will be too large to enable diversion of the river through the tunnel. That is why the location of the spillway would be in the riverbed and the use of it (without sills) for passing the water during this period is reasonable.

Thus the main type of construction of this project would be a concrete spillway with rock-fill abutments. The abutting dams should have central cores securing better junction with the concrete.

The rear abutments' edges should have a dip somewhat more than is demanded by their stability conditions. It is demanded for securing more safe junction of the dam with the spillway's concrete abutments.

It is desirable to flatten the steep right bank to provide for more proportional settlements of the rock-fill abutments. To the north-west of the Sandholt the ground level along a small stretch is some 0.5 meter lower of the proposed water storage level. Therefore a 2-3 meters high dike founded on lava will be required here.

The total dam's length along the top is 400 meters. The maximum height is 23 meters.

The rock excavated from the tunnel, bottom trench and canal can be used for the rock-fill.

In his Report Mr. Kjartansson (5) states the caution that in case of a high hydraulic gradient it is conceivable that some material might be washed out from the sedimentary layer between lava and basalt. Anything can hardly be said without studying the piping resistance qualities of these materials. Supposing that the lava consists of several layers and consequently its vertical permeability might be disregarded than the shortest leakage path the route of which would be around Sandholt is some 400 meters long. When the water storage would be full the average pressure gradient would equal 0.05-0.06. There might be no piping at such a gradient if to prevent the piercing of water into this interbed through the thickness of lava. Such a condition in lava might be attained after several years of dam's operation and in case of sedimentary deposits in the water storage bed.

Therefore we think that preliminary measures to prevent excessive leakage through the interbed should be provided for. A grout curtain might be constructed for this purpose in the foundation of the right bank dam to cut down into the interbed and exclude excessive water loss at the first stages of the operation period. Further more detailed investigations and studies might certainly prove that the compacted and deep curtain is not required here but nevertheless we deemed it necessary to envisage such measures at this stage and to include their costs in estimating dam costs.

Estimated volumes of main works for the dam:

-excavation of rock from spillway's bottom trench-canal and right's bank	88,100 cu.m.
-rock-fill in the dam-	3,500 cu.m.
-filters-	5,600 cu.m.
-spillway's concrete-	41,000 cu.m.
-grout curtain-	480 tons of cement
-gates and hoisting gears-	168 tons

The dam's cost with the spillway included was estimated at 1.6 mil$.

XI. THORISVATN DAM. (Dams on the Kaldakvisl and Thorisos rivers)

The Kaldakvisl river the tributary of the Thorisos river, flows to the north-west of the Thorisvatn lake at approximately 4-kilometer distance. The Thorisos river is an outlet from the Thorisvatn lake and after covering rather windy course extending for some 6 kilometers contributes its waters to the Kaldakvisl river. It is planned to use the Thorisvatn lake as a natural water storage with diversion of the Kaldakvisl river flow through the bed and the water storage of the Thorisos river.

Dams on both rivers would be required to carry out such diversion.

Icelandic engineers Messrs. Thoroddsen (4) and Kjartansson (5) proposed two alternative dam sites on these rivers that to our opinion are the most suitable of all other possible alternatives. One of them proposes to bridge both rivers by one dam. This dam site is located at a 2.5-kilometer distance from their confluence (Lower site). The other alternative is two seperate dams located farther upstream (Upper site). Owing to local relief lowering of the left river bank the Kaldakvisl water storage extends in the direction of the Thorisos river.

The Kaldakvisl river might be diverted to the Thorisvàtn lake through the Thorisos water storage by means of a connecting canal. The Thorisos water storage level is similar to that of the lake.

The results of geologic reconnaissance studies are given in Kjartansson's (5) and Nicol's (3) Reports.

In case the dam would be located at the lower site its foundation on a considerable length will be in palagonite formations and post-glacial lava underlain by sedimentary deposits of moraine and alluvium. All the rocks are highly permeable. The maximum dam height at water storage elevation 571 meters equals 26 meters.

The dam at the upper site would be founded on basalt and on moraine under the right abutment at high elevations if moraine is of considerable thickness. The dam's height in the riverbed section at water storage elevation 577 meters would be 22.0 meters. The Thorisos river at the proposed site location flows on moraine. Its right abutment would be on post-glacial lava of the similar quality to that at lower site.

Moraine is characterized as being rather tight and impermeable.

The foundation conditions at this site on the Thorisos river are the most investigated. Six boreholes were drilled that made it possible to outline on the cross-section the discovered layers to the depth of 20 meters. Therefore the data on the geology of this dam site are the most reliable in comparison with other sites. The dam's height here would not be more than 12 meters.

In comparing the two location alternatives it might be seen at this stage of knowledge of layers and their filtration properties that both sites are quite similar. The upper site does not seem any dangerous in relation to foundation and leakage-preventive measures might be kept to minimum if materials filled basalt joints are not apt to being washed out.

The dam foundation conditions at the upper site as well as at the lower site across both rivers are very much alike owing to the presence of lava. However other rocks at the site are of different qualities and the upper site at Thorisos seems more advantageous as the permeability of tight moraine might be less than that of the palagonite rocks at the lower site.

We consider the rock-fill construction to be more preferable if they are to be located at upper sites.

This type of construction is reasonable for the lower site as well since it mightn't be possible to obtain sufficient amounts of suitable moraine or alluvium within close distance. The required volume of rock for the dam should be excavated from quarries.

For alternatives cost estimation comparison we calculated volumes of work for the two alternatives with the data following. The dam at the lower site has the top's elevation 574 meters(water storage elevation 571 meters), the dam's length is 435 meters, the top's elevation of the Kaldakvisl dam 580 meters (water storage elevation-576 meters), the dam's length is 940 meters. The latter was defined by two conditions. The first consisted in securing the raising of the Kaldakvisl water storage level in order to carry out river diversion at the piezomteric gradient of canal waters. The second to provide volume of rock excavation from the tailrace canal equal to the volume

of the rock-fill in the dam taking into consideration the porosity of the materials. While the first condition was stipulated by technical requirements, the second by purely economic considerations.

As the result of estimating total volumes of dams at the above-mentioned data the following is obtained:

Upper dam site

Kaldakvisl dam-	203,000 cu.m.
Thorisos dam-	65,500 cu.m.
Total-	268,500 cu.m.

Lower dam site

One dam across Kaldakvisl and Thorisos- 510,000 cu.m.

Taking foundation treatment costs as being approximately the same the alternative with the dams location at the upper site should be considered more economically reasonable.

The spillway would be required for construction discharge from the Thorisvatn lake. It might be located either on the lake's bank or in the dam on the Thorisos river. The definite location can be selected later in connection with the layout of construction works. The proposed by us location of spillway envisages its usage in an incompletely built condition (without sills) for diversion of the Thorisos and Kaldakvisl during construction period. When the Kaldakvisl dam would have been built it certainly won't be the only solution and it might turn out that the spillway located on the lake's bank would be economically more justified.

The spillway is calculated to pass 500 cu.m./sec. at the normal water level and is composed of two spans by 15 meters with the gates 5 meters high. Its foundation is on tight moraine.

Cross-section surface is defined by the condition that the velocity of water in canal at the average Kaldakvisl river discharge of 60 cu.m./sec. should not exceed 0.6 m/sec. The sluice should be installed at the canal's end section. We thought it to be extremely desirable to periodically wash the canal and water storage that would be deposited by bedload. Such washing would enable to form another bed of the Kaldakvisl river for the time when the rest part of water storage volume would be filled with bedload. At the same time it won't be

necessary to build on the dam of this water storage.

The building on of the Thorisos dam nevertheless would be desirable, after several years of operation and the bedload got stored up, in order to raise the Thorisvatn lake level.

Unlike the dam foundation on the Hvita river dams would be founded on post-glacial lava here.

Several flows are expected here. Lava is sufficiently tight and firm for the rock-fill construction but it is of high permeability in horizontal direction which can be explained by the considerable cavernousity of the lower part of the flow and as is supposed by Icelandic experts by the high permeability of sedimentary interbeds between flows. We came across such foundations during the design work for the dam on the Razdan river (Armenia). At the first stages of the design work for Arzninskaya power plant we apprehended considerable leakage and piping through intercalations between lava flows composed of sedimentary materials and scoria. However the more detailed investigations and studies proved them to be several times less permeable than lava. Thus it was possible to lessen the depth of the grout curtain carrying it down to the first interbed between flows. The ten year exploitation experience and observations confirmed the rightness of the solution on anti-leakage foundation protection of the Armenian hydro-power plant. Thus it might be expected that in this case anti-leakage curtain on lava section may turn out to be rather small.

When selecting the method of cutting down horizontal leakage paths they are always apprehensive of considerable cement injection expenditures and of the curtain to be costly and make a solution in design to cut down a wall cut-off. But as a rule grouting is adopted later on but not the wall cut-off. If to take the permeability of lava at the area equal to several hundreds of 'Lugeons' then on the basis of grouting experience in similar conditions it might be supposed that expenditure of concrete would not exceed 200 kilogramms per running meter at approximately the same expenditure of clay being constituent of grouting mixture.

It should be taken into account for foundation treatment that the uppermost lava layer sealed by small-size wind-blown and water--driven materials is of low permeability. It should be left as it is.

It should be noted when working out anti-leakage measures that lava is piping resistant. The measures should be taken to prevent the stripping off the upper layer during construction.

As the experience shows the further investigations reveal worse quality of foundations than was thought at the preliminary stages therefore we deemed it necessary to envisage grout curtain in the dam's foundation on the Kaldakvisl river as well as on the Thorisos river.

At the above-stated preconditions the estimated volumes of main works for the dams are as follow:

Kaldakvisl dam (water storage elevation 576 meters)

-excavation of moraine-	32,000 cu.m.
-rock-fill-	155,000 cu.m.
-core-	25,000 cu.m.
-filters-	41,000 cu.m.
-grout curtain-	250 tons of cement.

Thorisos dam (water storage elevation 571)

-excavation of moraine-	15,600 cu.m.
-excavation of rock-	30,000 cu.m.
-rock-fill-	48,000 cu.m.
-core-	9,000 cu.m.
-filters-	14,000 cu.m.
-concrete and reinforced concrete of the spillway and canal's sluice-	7,230 cu.m.
-grout curtain-	550 tons of cement
gates and hoisting gears-	42 tons.

Considering that the whole volume of rock and moraine excavations from the diversion canal would be used in construction of both dams the estimated cost would be 1.43 mil$.

In view of the advisability to raise the Thorisvatn lake's water level we considered the possibility of building Thorisos dam up to elevation 579 meters. The feature is aimed to obtain more proportional energy production at the Burfell power plant. In this case the both water storages would have the same elevation 579 meters and the regulating sluice won't be required.

Since the volume of work for the Thorisos dam would increase the estimated cost of the two dams would be 1.92 mil$.

A certain raising of the Kaldakvisl water storage level will be required for creating piezometric gradient at the diversion canal. Hydraulic investigations can clarify with the figures of this raising. But at the relatively small length of the canal raising by one meter would probably be sufficient. Then Kaldakvisl dam would cost approximately 0.3 mil$ more.

XII. TUNGNAARKROKUR DAM.

The project is located on the Tungnaa river which is the largest tributary of the Thjorsa river. The dam site location was selected slightly upstream from the gorge where river level is 460 meters. Upstream from the dam site there is a broad overflow covered with clay testifying to the existance of the ancient lake here. The river flows on palagonite rocks forming also the steep right bank.

The left bank rocks are represented by several lava flows and consequently the bank is very steep. When nearing the riverbed the thickness of lava is lessening on account of the higher lying of palagonite roof overlain by lava.

At the adopted for this project water storage elevation at 499 meters the dam's height in the riverbed section would achieve 47 meters.

From the standpoint of geology and topography a concrete gravity structure as well as rock-fill one are feasible here in view of using rock from tunnels and canals excavations. A concrete structure at such height would certainly be more costly than rock-fill one. Therefore we chose the latter type of construction. The available data on the construction materials at site though they are weakly investigated so far do not allow to hope on the location of materials suitable for the core. Clay of lake deposits is too 'fat' to be suitable for these purposes.

Thus we found it possible to propose for this dam site a fill type construction with the reinforced concrete membrane. The membrane should be an elastic one with gravel support. When speaking of the elasticity we mean establishing not only of vertical joints but horizontal as well with impermeable contractions made of rubber.

As was the case with the recommended above rock-fill dam with the sloping ground core this type of construction makes possible to carry out dumping of rock-fill independently of gravel treatment and concrete membrane.

The proposed freeboard of 3 meters might possibly be lessened in more detailed design work when constructing concrete wave standing.

The cut-off with elastic junction is envisaged in the dam's foundation. The depth of the cut-off in the riverbed section should be not less than 2.5-3.0 meters. The grout curtain should be carried down under the cut-off. At the section where moberg floors the foundation the curtain's depth depends not only on the lessening of permeability with the depth but also on the kind of underlying rocks. It should be strived at cutting down this curtain into breccia formations if the layer turns out to be connected with the water storage directly or through another layer with high permeability.

In relation to piping the most dangerous might be sedimentary interbeds cropping out to the surface at the water storage zone. We think it possible to apply two alternative types of grouting measures here. The first one-carrying down the grout curtain under the dam and the second-dumping of lake clay at the dam's section where sedimentary rocks outcropped to the surface. The length of such blanket can be defined only after detailed foundation investigations and certain field tests. In speaking of leakage through lava it should be noted in proposing leakage-preventing measures to mind that allowance of considerable gradients is not dangerous for lava and certain water loss during first stages of the period when water storage is full should lessen in the near future. The measures should be taken to protect ground of the trench cut-off from the washing out of materials at contact zones. Compacted grout curtain would be certainly required under the dam in lava close to the riverbed where the dam's height would be some 30-40 meters. Further at the dam's section with small height founded also on lava the grout curtain might be uncompacted or even be substituted for the trench cut-off.

The spillway is located on the right bank near the intake that allows to discharge ice and sludge ice.

The spillway is calculated to pass high flow up to 500 cu.m./sec. and is composed of two spans by 12 meters and the nearest to the intake span is equipped with flap gate the other with tainter gate.

At the dam top's elevation 502 meters its length is 900 meters. The upstream slope is 1:1,4 downstream is 1:1.5

The estimated volumes of main works for the dam are as follws:

-excavation of rock-	32,000 cu.m.
-rock-fill-	300,000 cu.m.
-concrete and reinforced concrete-	11,500 cu.m.
-grout curtain-	650 tons of cement
-gates and hoisting gears-	30 tons

The estimated dam cost is 1.83 mil$.

XIII. HRUNEYJAFOSS DAM.

The dam of this project would be located at a 3-4 kilometer distance from the falling of the Kaldakvisl river into the Tungnaa river. Mr Thoroddsen proposes in his Report (4) two rather suitable dam sites with the left bank abutments at the same place i.e. on the northern slope of the Hrauneyjafell mountain. In both alternatives the dam has the maximum height and volumes that could have been obtained for this project.

We have considered only one dam alternative at the lower site where the dam has shorter length since the axes of these two dam alternatives are located in a short distance from each other and the difference in dam volumes evidently very slight. The river water elevation here is 413 meters.

From the engineering-geological standpoint the foundation condition are similar to that of the Tungnaarkrokur project. The considerable part of the dam would be on the left bank floored by several flows of post-glacial lava some 40 meters thick. (3). Palagonite moberg underlying it consists of pillow lava with interbedded sands and gravel most of which appear compact. The Tungnaa river has at this section two branches cut down their riverbed in the upper lava flow. Near water's edge and farther on the right bank crops out the above-mentioned palagonite rock. No investigations have been carried out on the permeability of lava, contact zones and moberg.

There is ample amount of materials for rock-fill, and the aggregate. Materials suitable for core and filters of the rock-fill construction were not found.

At the proposed for Hrauneyjafoss water storage elevation 425 meters the average dam's height won't exceed 9 meters. Proceeding from the above-said we considered two dam alternatives feasible under these conditions i.e. rock-fill and concrete gravity structures.

The rock-fill construction for the sake of comparison was adopted with the reinforced concrete membrane and slopes 1:1.4 and 1:1.5. The concrete construction with downstream edge slope 1:0.75. Buttresses are coming over this edge and serve as spans for the road bridge.

The dam volumes were estimated for cross section on the basis of 1:5,000 topographic map. The estimated costs of the two dam alternatives are practically the same-about 2.5 mil$. We recommended for this project a concrete gravity structure as more preferable taking into account that a concrete spillway will be at the dam's site which will be dwelt on later.

The foundation treatment would consist of stripping off the upper bubble lava layer that was taken for our estimates as being one meter thick. Furthermore it would be necessary to excavate lava for cutting down the dam's cut-off near the face edge. The grout curtain under the total dam's length probably would not be required in view of the small height of the dam. However it would certainly be necessary in the riverbed and adjoining sections to eliminate the possibility of leakage along short paths mainly through contact layers and lava. When carried down into lava the grout curtain might not be compact but in cases contact layers would appear apt to washing out the deepening of the curtain would be required down to the point where control water absorption is equal to 0.05-0.10 liters/min.

The depth, the length, number of rows etc. might be definitely established only after testing or during the construction.

But as was said in connection with other projects contact layers somitimes turn out to be less permeable than lava. In this case the curtain may be carried down only to contact layers as it is often done in practice.

For estimating costs of anti-leakage foundation-protective measures we adopted grout curtain with the following characteristics: the left bank section extends (riverbed included) 1400 meters; average curtains depth roughly equals the thickness of the upper lava flow i.e. 10 meters; distance between boreholes-3 meters; cement expenditure-275 kilogramms per running meter of boreholes.

The right bank curtain is double-row, 250 meters long, average depth is 8 meters, boreholes at a 3-meter distance, cement expenditure-200 kilogramms per running meter.

The spillway might be located either in the riverbed or on the right bank near the power plant's intake that would be more reasonable

from the operation standpoint. However relatively considerable water discharges that should be diverted during the construction period would probably stipulate the division of the spillway into two parts each one of which would be located in the branches. At the same time the power plant's intake should be located possibly close to the spillway of the right branch. Therefore we found it possible to adopt the latter alternative and to divide the spillway consisted of two spans by 15 meters into two parts as was just stated. The spillway's tainter gates are 5 meters high. Both spans are capable to pass the design flow up to 1500 cu.m./sec. at normal water storage elevation. The spillway nearest to the intake might be equipped with taintor gate with flap or with fish-belly gate for passing ice.

To divert waters of one branch into another during the construction of spillway's piers the diversion canal would be required.

This canal might be used at the second stage of construction for diverting the left branch in the riverbed of the right one for passing the whole river flow between the piers of the incompletely built spillway.

Estimated volumes of the main works for the dam are the following:

excavation of rock from the foundation-	8,000 cu.m.
concrete of dam and spillway-	67,500 cu.m.
grout curtain-	1500 tons of cement
gates and hoisting gears-	109 tons

Estimated dam cost with spillway is 2.6 mil$.

XIV. BUDARHALS DAM.

It is planned to locate the Budarhals power plant on the Tungnaa river in 7.5 kilometers upstream from its confluence with the Thjorsa river where river elevation is 306 meters. The left bank as it is the case for the designed projects on the Tungnaa river is flat and formed by young basalt lava flows. The right bank is steep and composed of palogonite formations mainly of pillow lava that in Mr. Nicol s opinion (3) might be rather tight. Ground moraine layer 5 meters thick is reported on this bank. The considerable part of a dam would be founded on the flat left bank where there are no sedimentary deposits and the surface layer 1-2 meters thick is a scoriaceous lava layer. Rocks in the riverbed section can be judged on the basis of the map contained in Mr. Kjartansson's Report (5) where it is shown that palagonite rocks of the right bank floor the riverbed.

The construction materials for the dam include basalt excavated from tunnels and the spillway's bottom trench with canals and also moraine suitable for the impervious core according to Mr. Nicol's Report.

At the water storage elevation established for this dam at 325 meters the maximum dam height would be 24 meters. Taking into account that the volume of basalt excavated from the tunnel and canals would be sufficient for rock-fill construction and the availability of moraine suitable for the core we think this type would be the most reasonable. The spillway at least part of its spans would be located in the riverbed. Therefore the ground core of the dam should be vertical or slightly sloping.

Slopes at the preliminary stage of design work might be taken roughly as the upstream 1:1.7, the downstream 1:1.6. The total dam length is 680 meters. The junction of the dam with the foundation on the left bank might be carried out by construction the trench cut-off only cutting down the upper layer of scoriaceous lava, Other measures aimed at lowering the pressure gradient might not be required if sedimentary materials won't be found later in the foundation.

In view of the small dam's height scoriaceous lava might'nt be stripped off from the rock-fill dumping area.

It would be advisable to strip off moraine in the dam foundation in case its layer does not exceed one meter. However it won't be reasonable if the thickness is much greater. The inverted filter should be placed at the contact zone with the rock-fill to prevent possible washing out of moraine.

The spillway adopted by us for estimating volumes of work is calculated to pass flow up to 2500 cu.m./sec. It is composed of seven spans by 15 meters and tainter gates 5 meters high. Its best location from topographic standpoint would be on the left bank since the volume of the required excavation would be less here than on the right bank. However from the operation standpoint in case the power plant's intake is located on the right bank it would be better to place the spillway there close directly to the intake.

The compromise solution would be the location of the part of spillways spans at right bank's high elevations and the other part in the riverbed section. The number of spans to be located in the riverbed section would be defined by the conditions of passing construction flow after damming the river.

We suppose that this solution would be the most economic and probably the inevitable one since there won't be any possibility to divert the river at this section during construction period through the tunnel at moderate costs. However the proposed construction layout would require the widening of the riverbed by shearing the left bank to secure the allowable limits of the riverbed narrowing by abutments. Nevertheless the total volume of excavations should be less than if the whole spillway is located in the riverbed section or on one of the banks. Bottom openings should be envisaged in spans' sills for washing the storage.

The above-mentioned grout curtain in this case should be extended under the foot of the whole spillway and partly under the length of the right rock-fill dam's abutment with the junction with its core. To estimate roughly grouting costs the length of the curtain was adopted to be 400 meters.

The permeability of lava and palagonite was considered to be the same. The curtain is single-row with a 2.5 meters distance between borehole and the cement expenditure about 200 kilogramms per running meter of a borehole.

All the above-stated measures are proceeding from lessening of filtration loss through the foundation after short period of operation of the water storage on account of the sealing of foundation.

Volumes of work for the dam with the spillway and spillway's canal excavations included estimated as the following:

-excavation of moraine-	24,000 cu.m.
-excavation of rock-	325,000 cu.m.
-rock-fill	164,000 cu.m.
-core-	33,000 cu.m.
-filters-	37,000 cu.m.
-concrete for spillway-	22,000 cu.m.
-grout curtain-	320 tons of cement
-dates and hoisting gears-	58 tons

Estimated dam cost-1.7 mil$.

XV. SOME ASPECTS OF THE ROCK-FILL DAMS CONSTRUCTION ON THE THJORSA AND HVITA RIVERS.

This chapter covers some aspects of the site organization to avoid repeating them each time since they are of general character for all or at least for the part of the above considered dams.

The height of all dams in the Hvita and Thjorsa river basins is small and in fact almost any kind of volcanic rocks can be used for dam construction. The exception might be conglomerates of palagonite series that would require considerable expenditures on their treatment.

It is well known that the size of stone all other conditions being similar depends on the method of its excavation. For the most of the dams the stone would be excavated from tunnels with an area of 50-m^2 section (profile) and would contain great amount of small-size stones. The rock for dams with ground central or vertical core might be transported for dumping on the dam without any preliminary treatment.

The thickness of lifts which usualy increases with the size of stone for the greater part of the considered dams would probably be rather small. In case of dumping the rock excavated from canals by quarry method only it might be dumped on dams at its full height or in two storeys.

When dumping the rock on dam sections up to 15 meters high the compaction by transport means only might be sufficient with the sluicing with water using hydromonitors. The lower dam storeys where their height would exceed 20-25 meters and when small-size stone would be used additional compaction might take place by rollers and vibrators. These means might be used in dam construction during the winter season when the sluicing would be impossible.

These details can be cleared out only during the construction period after stone-size tests of the rock excavated by the methods which would be used in actual dam construction and after experimental dumping of the rock on the fill.

The greatest difficulties might be expected in obtaining the required compaction of the core of moraine. This material as a rule has

restricted moisture limits up to which it can be compacted to the greater value of its volume-weight. Numerous experiments on moraine of the Kola Peninsula when constructing power plants on the Niva river in the U.S.S.R. proved that the moraine moisture interval allowing to obtain the greatest volume-weight ranges from 5 up to 8%. When it achieves 10-12% moraine transforms into floating earth that is absolutely impossible to compact. It is evident that the material was of the disturbed structure and boulders oversizing 100 mm were seperated. In natural conditions moraine's moisture usually exceeds the optimum one. That is why the fill of sufficient compactness can not be created without drying the moraine prior to its compacting. It mightn't always be possible to carry out in areas with heavy precipitation. During recent years placing of moraine is performed by one-meter thick lifts without seperating of boulders. It is compacted mainly by smooth vibrating rollers. The number of passes is defined through the tests. The volume-weight required by the design serves as the criterium in this case. The loess deposits are used as well for the impervious elements of the rock-fill dams in the Thjorsa river basin. The loess being devoid of the above-mentioned disadvantage can be used at a greater range of its moisture.

In view of difficulties in placing of moraine in the conditions of damp climate the hydraulic method of moraine compaction is being applied on a wide scale beginning from 1948. Moraine can be placed in any weather when this method is applied even at 15-20 degrees below zero. The quality of the fill made according to this method as is proved by 15 years experience of operation and tests is quite good.

The most recent project where this method was applied was the riverbed section of the Upper-Tulom power plant's earth dam built by Finnish and Soviet engineers on the Tulom river in the period of 1962-1964.

The main features of this method is dumping of materials into still water of ponds, created by cofferdams (for the lower storey) and by dikes (for upper storeys). When dumping materials in storeys located higher river than elevation the water is pumped into those ponds.

The storey height was some 3-4 meters for the first dams and 12.5 meters in case of the Upper-Tulom dam. Fencing dikes are built not

up to the full storey's height but built upon as the fill works proceed and its height increases.

The upper part of dikes as a rule is higher some 0.5-0.7 m. than the pond water elevation. Materials are dumped by tip-up lorries on the slope formed by the previously dumped rock. Moraine is not sorted out before being loaded on those lorries seperating only those boulders that oversize excavator's scoop. The dumping is usually carried out by lifts being parallel to the dam's axis the width of lifts being some 10-15 meters. The water squeezed out by the dumping is diverted through the 8-10-inch pipe.

When dumping cloddy moraine into water it becomes loosened with particles of smaller size. The preliminary compaction is performed by tip-up lorries and bulldozers when the material is dumped into water. The raising of the fill over the pond water level is more frequently adopted at 0.5 meter and is defined by avoiding the sinking of trucks'wheels into the water.

The similar method has been used in dumping the core and membrane of some other rock-fill dams in the U.S.S.R. (e.g. Iriklinskaya power plant's membrane).

We consider this method of placing moraine in the core of rock-fill dams as suitable for Icelandic conditions.

The dumping of moraine into water might considerably simplify the construction works when building the riverbed sections of the rock-fill dams. The simplification would include the avoidance of the grout curtain under the riverbed section of a dam as well as the drainage of the foundation trench in case the lower storey is carried out by this method. The bouldery cloak covering the river bottom was not stripped off in the construction of several dams by dumping materials into water. The moraine fill as the explorations showed has better contact with the uneven foundation then moraine dumped by dry method. The filling in of the bouldery layer with moraine flattens the dam's foundation after the storage is full.

Some possible riverbed section designs of the rock-fill dam are shown on the drawings attached to the Report.

In this case the materials are dumped into still water and upper and lower benches are constructed in the riverbed for this purpose. The lower bench is always included in the dam's body while the upper one increases the leakage path and might be slightly seperated. The distance between the cofferdam and the dam's upstream slope is filled in with moraine dumped into water.

The same drawing shows the core construction by 'wet' method.

As can be seen the proposed method of the vertical core dumping means the simultaneous dumping of the rock-fill by horizontal lifts of rather small thickness that is inevitable as well with the 'dry' method of the vertical core dumping.

In case of the sloping core the two dumpings can be carried out independently of each other by 'dry' as well as by 'wet' method.

The use of the above-described method of constructing of dams or their impervious cores of moraine at all the Soviet projects proved to be economically justified and was stipulated by climatic and weather conditions.

Other general aspects of the site organization in rock-fill dam construction in the Hvita and Thjorsa river basins we haven't dwelt upon to shorten the volume of the report as well as in view that they are extensively covered in the western technical books and periodicals.

XVI. CONCLUSIONS.

The present Report gives consideration to the conditions of dam construction, outlines the argumented recommendations on types of dams and defines volumes of main works as well as estimates costs of the two projects on the Hvita river and of the ten projects on the Thjorsa river and its tributaries. Furthermore the dam foundation protective measures and those aimed at diminishing water loss on account of leakage through the foundation rocks are proposed herewith as far as it was possible on the basis of geology exploration data available at the time of writing this Report.

When selecting the dam alternatives the due consideration was given to the peculiar project's conditions as well as construction conditions of building the dam structure itself and to general aspects of the site organization for the appurtenant features closely related to the dam.

The maximum heights of the considered 12 projects range from 13 up to 44 meters. The greater part of these dams would be built at the diversion type power plants with penstocks. All the tunnels would be in the rock. The spillways with head and tailrace canals located at dams would be also founded on the rock.

The Report gives the reasoning on the suitability of rock-fill structures for the majority of dam sites. The rock-fill dam structure with the ground sloping core responds to the greater extent to the natural conditions of Iceland that were covered in the first chapter of our Report. In case the slope of the downstream edge of such a core equals the angle of the rock-fill's natural slope the latter might be carried out independently of the ground core dumping. It enables to perform the stone dumping under any weather conditions while the core and filters are being dumped under the most favourable conditions when the greatest fill compaction might be obtained.

However in cases where the spillway is situated within the dam's length dividing it into two parts dam with the central core type was recommended. Such dam construction is desirable when it secures better junction of the dam with the concrete of the spillway. The dam constructions with vertical cores were found to be more economical than those

with the sloping core that require the flattening of the upstream slope and greater amounts of materials though the former are less suitable for natural and construction conditions of Iceland.

Moraine might be used for the core material as well as loess available in the vicinity of dam sites.

In the exceptional cases when suitable local construction materials for the core and filters haven't been found the rock-fill construction with the reinforced concrete membrane was recommended (the Tungnaarkrokur dam) the type that meets the seismic requirements of Iceland.

The Hrauneyjafoss dam with the top's height of 10 meters as the compared estimates showed would have the same price for the concrete gravity alternative as well as for the rock-fill one. Because of some considerations of the construction nature the preference was granted to the concrete gravity alternative the more so in view that the geology of the foundation is quite satisfactory for this type of structure.

At the majority of the considered dam sites the foundations are on volcanic rocks. However in some cases the bank abutments would be founded on glacial moraine when its thickness is considerable. Old basalt flows are of low permeability and the leakage-preventive measures are aimed at not allowing to wash out sedimentary materials from joints. Such rocks as a rule crop out in the riverbed gorges cut down by the river stream.

The grout curtain construction cutting it down to the in fact impermeable rock for the given head on the dam was recommended to lessen the pressure gradients. The degree of the rock permeability is defined through specific water absorption tests. It is not recommended to use alongwith the permeability criteria proposed some tens of years ago by Lugeon since it results in the unjustified increase of grout curtains costs. In the Soviet construction practice this criterium for the dams up to 50 meters equals 0.05-0.1 litre/min (0.01 litre per min. according to Lugeon).

Almost all the dams on the Thjorsa and Tungnaa rivers would have foundations on post-glacial basalt lava. The rock is sufficiently firm and piping-resistant but highly permeable. The leakage-preventive

measures are aimed at the compaction of the rock to diminish the water loss which might be rather considerable after the head is created. The construction of usual grout curtains might turn out to be rather costly. However the Thjorsa river waters carry great amount of sedimentary load that falling down in the riverbed creates the impermeable cover.

The Report recommends to take an advantage of this peculiarity of the Thjorsa water passage. The results of the field exploratory work carried out by H. Tomasson, the Icelandic geologist, testify to the reality of the supposition that in some may be very short period of time dam foundations on lava would get sealed. Nevertheless incompact single-row grout curtains should be envisaged to prevent through washing out of sedimentary deposits.

The main part of the rock-fill would be carried out in stone excavated from the borrow-pits without any sorting out. The technology of dam construction would depend greatly on the size of the stone. It might be expected that the stone from tunnels would not be large--size and its dumping should be performed in lifts of small thickness. However the stone from canals excavated using the quarry method is expected to be of large size and the dumping in pretty thick lifts would be advisable might be even up to the full dam's height. In both cases the required compaction of the rock-fill might be obtained only by the sluicing with water and using the transportation means. The additional compaction would be needed only at the core and filters adjacent zones where dumping in lifts 2-meters thick is compulsory.

For some dams (e.g. the Hvitarvatn dam) the amount of stone excavated from borrow-pits won't be enough. The needed rock would have to be excavated from special quarries. However some alluvial deposits might be found within the reasonable distance and their excavation would certainly be cheaper than that of the rock and even of the moraine. That is why it is recommended to use this material for dumping into the central part of the dam as was the case with the Holjes dam constructed in Sweden.

Some difficulties might arise when dumping moraine into the dam's

core because of the relatively heavy precipitation in Iceland. As was already mentioned above the main dam type with the sloping core contributes to a certain degree to overcoming of these difficulties. Nevertheless some of these difficulties are caused by the fact that moraine moisture in natural conditions considerably exceeds the optimum and its compaction even under dry conditions would not give the desired effect. The Report contains the recommendations on the so-called 'wet' method of moraine placement extensively used in the U.S.S.R. Furthermore this method is proposed for the lower storeys of the dams' riverbed sections that allows not to construct any grout curtains at these sections and to do away with the drainage of the bottom trench between cofferdams. The recommendation is accompanied with drawings attached to the Report's appendices.

All the main dam characteristics as well as estimated volumes of work and costs of the considered dams are represented in tables also included in the just-mentioned appendices.

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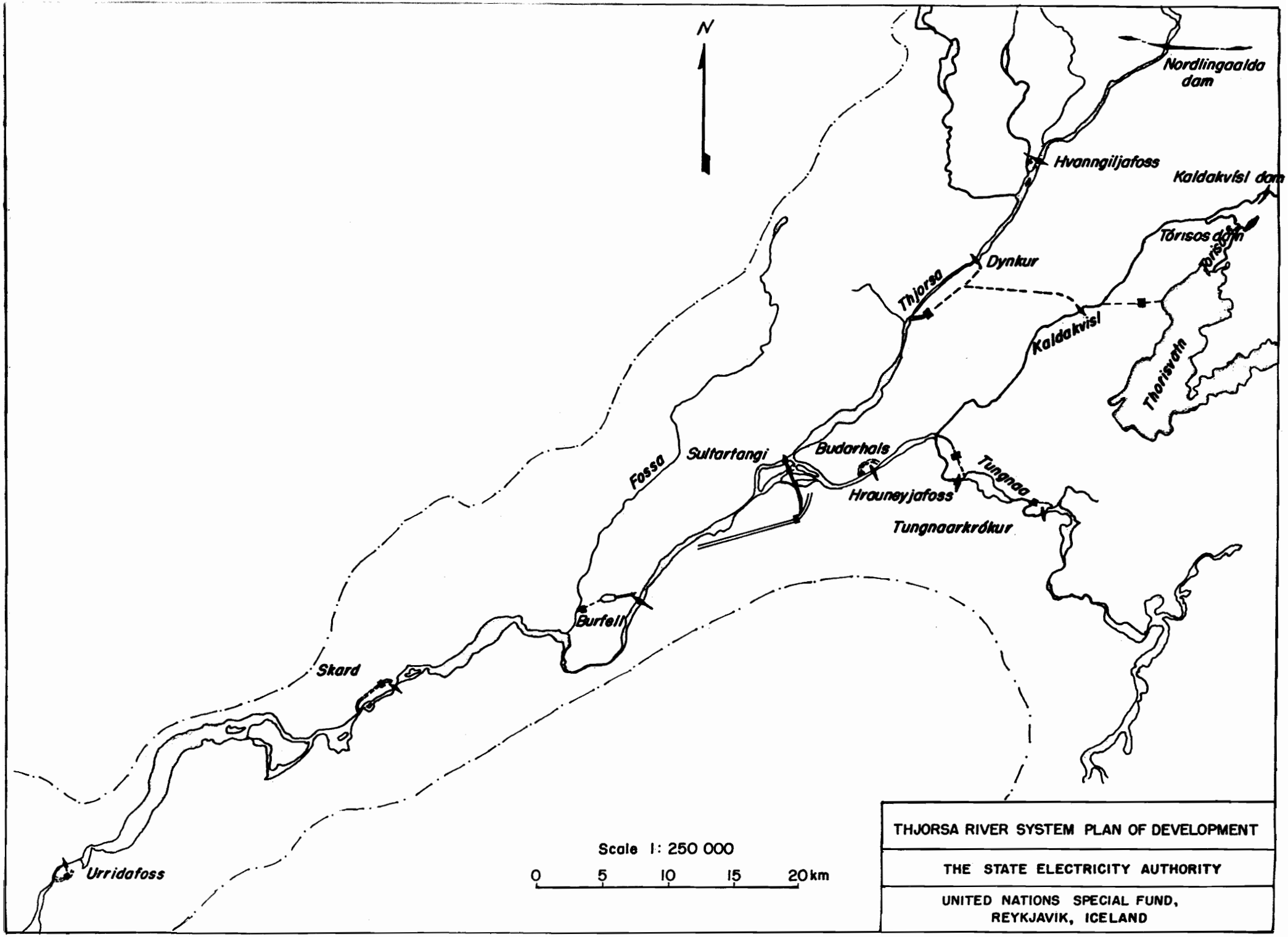
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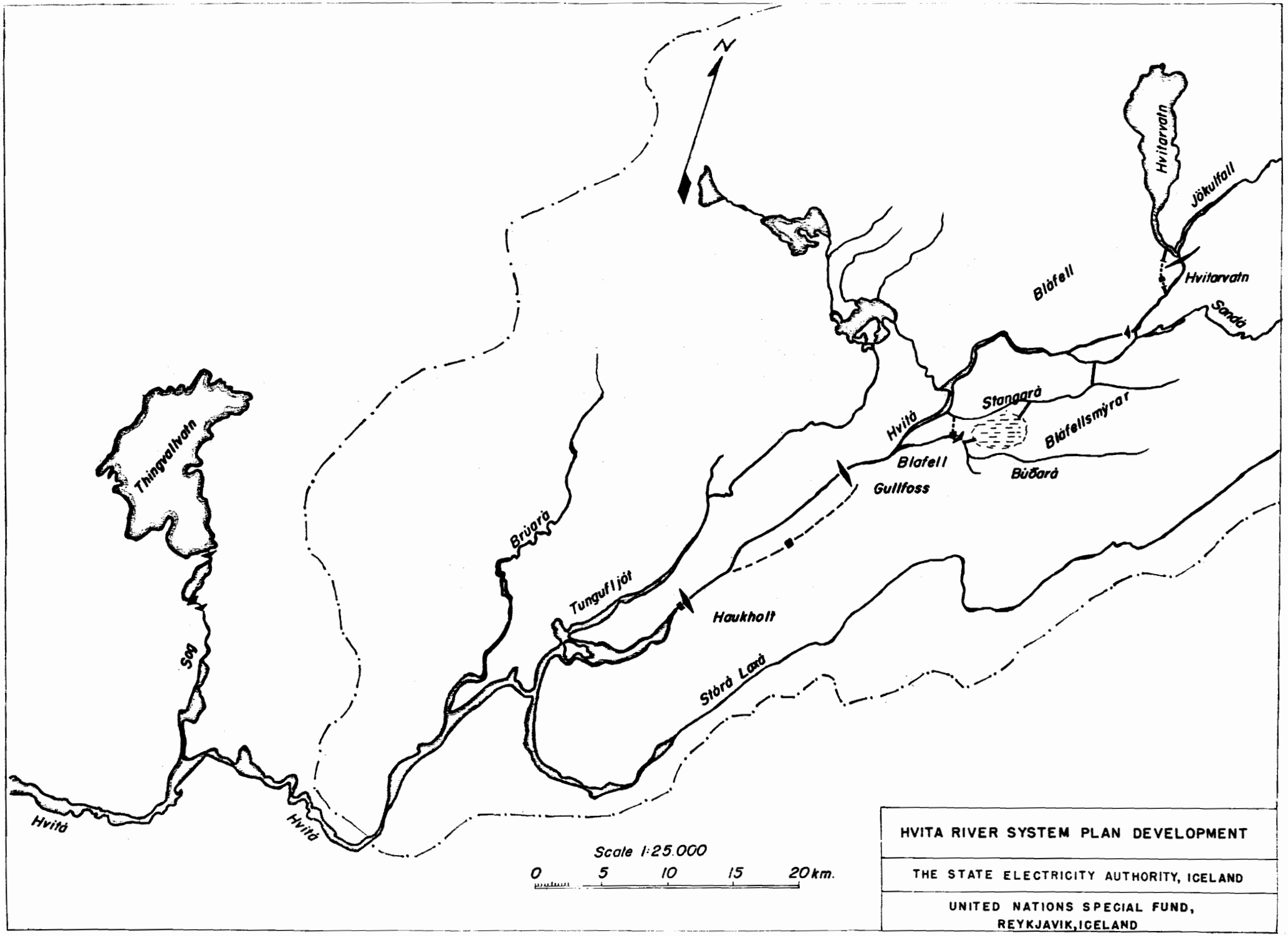
Budarhals
Tungnaa
Hrauneyjafoss
Tungnaarkrókur
Burfell
Skard
Urridafoss
Scale 1: 250 000
0 5 10 15 20 km
THJORSA RIVER SYSTEM PLAN OF DEVELOPMENT
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND,
REYKJAVIK, ICELAND


N
Hvitarvatn
Jökulfall
Hvitarvatn
Sandá
Blafell
Stangará
Blafellsmýrar
Hvitá
Blafell
Búðará
Gullfoss
Þingvallvatn
Brúará
Tungufljót
Haukholt
Sog
Stóra Laxá
Hvitá
Hvitá
Scale 1:25.000
0 5 10 15 20 km.
HVITA RIVER SYSTEM PLAN DEVELOPMENT
THE STATE ELECTRICITY AUTHORITY, ICELAND
UNITED NATIONS SPECIAL FUND,
REYKJAVIK, ICELAND

APPENDICES.

App. 1. UNIT COSTS.

App. 2. HVITARVATN DAM PLAN.

App. 3. HVITARVATN DAM SECTION.

App. 4. GULLFOSS DAM PLAN AND SECTION.

App. 5. NORDLINGAALDA DAM PLAN AND SECTION.

App. 6. HVANNGILJAFOSS DAM PLAN AND SECTION.

App. 7. DYNKUR DAM PLAN AND SECTION.

App. 8. SULTARTANGI DAM PLAN AND SECTION.

App. 9. URRIDAFOSS DAM PLAN AND SECTION.

App. 10. LOCATION OF DAMS ON KALDAKVISL AND THORISOS RIVERS, DIVERSION CANAL AND SECTIONS.

App. 11. DAMS WITH IMPERVIOUS ELEMENTS OF MORAINE DUMPED IN WATER.

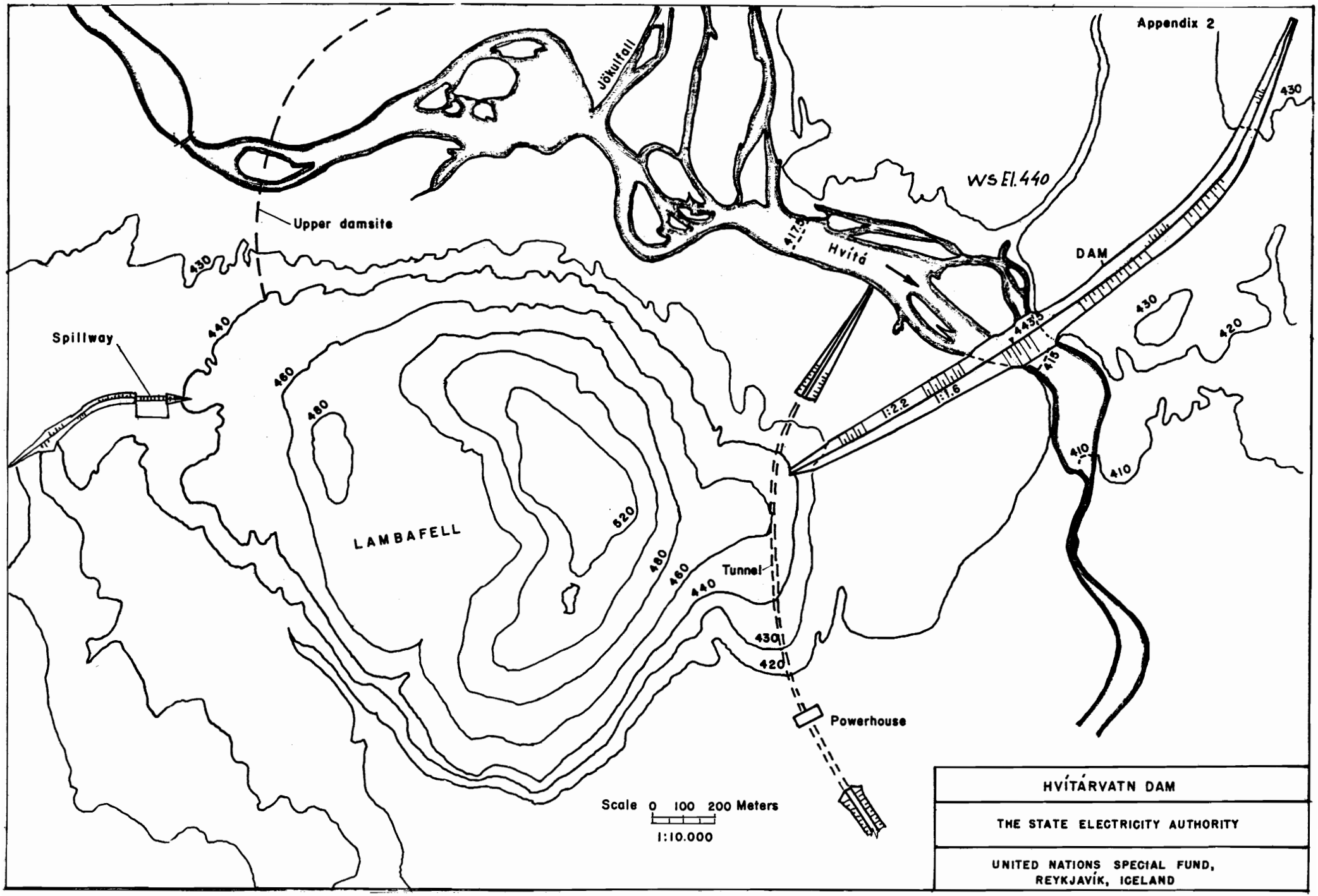
App. 12. HVITA AND THJORSA RIVER DAMS PROJECT DATA.

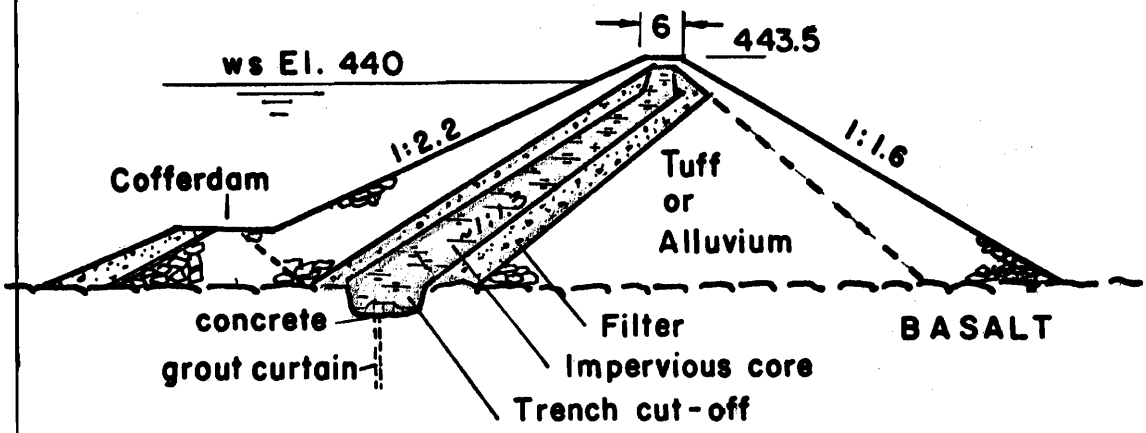
App. 13. THJORSA AND TUNGNAA RIVER DAMS PROJECT DATA.

UNIT COSTS

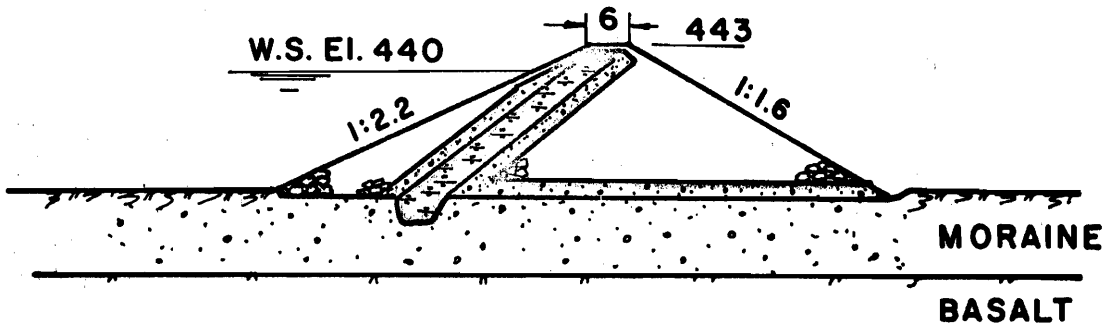
used in cost estimates of dam alternatives.

1. Rock excavation from quarry by blasting, haul distance 1 km, cost per m^3 -	4$
2. Moraine excavation, transportation to construction site, compaction etc., cost per m^3 -	3.2$
3. Filters excavation and transportation to construction site of one m^3 of	
-natural mixture-	2-3$
-processed mixture-	6-7$
4. Rock excavation from canals by blasting, haul distance 1 km, cost per m^3 -	4$
5. Moraine excavation from canals, cost per m^3 -	1.5$
6. Excavation of loose materials from canals, cost per m^3 -	1.0$
7. Excavation of rock from narrow foundation trenches, loosening, haul distance 1 km, cost per m^3 -	10$
8. Excavation of loose materials from narrow foundation trenches, haul distance 1 km, cost per m^3	3$
9. Steel structures	
penstocks, cost per ton -	600$
gates, cost per ton -	800$
hoisting gears and other appurtenant equipment -	30% of the gates cost
10. Concrete works	
-concrete mixture, cost per m^3-	24$
-forms, cost per m^3-	4.7$
11. Fitting	
-in rods, cost per ton-	170$
-cutting, welding etc., cost per ton-	75$
-welded fitting in structure, cost per ton-	245$
12. Foundation treatment, cement, cost per kg -	0.10-0.15$

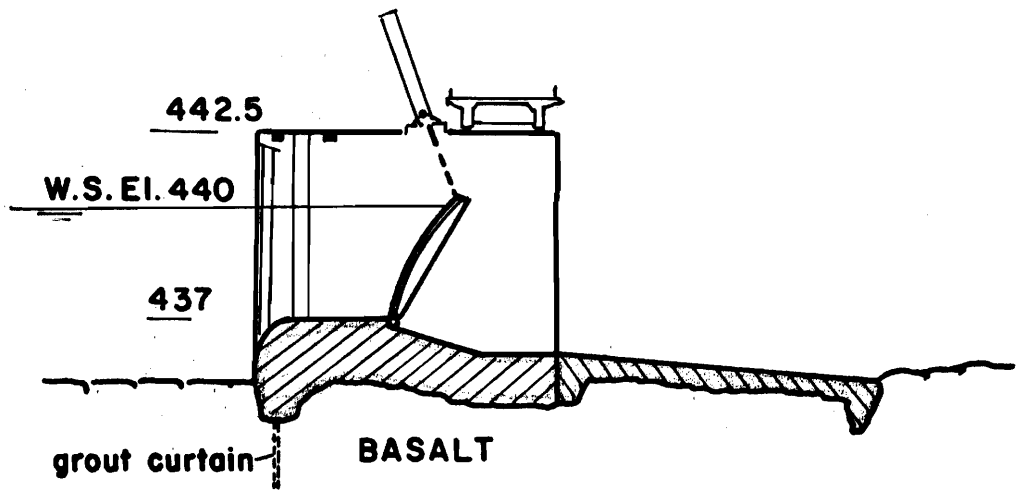

Jökulfall
Upper damsite
WS El. 440
417.5
Hvítá
DAM
443.5
415
Spillway
LAMBAFELL
Tunnel
1:2.2
1:1.6
Powerhouse
Scale 0 100 200 Meters
1:10.000
HVÍTÁRVATN DAM
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND,
REYKJAVÍK, ICELAND



RIVER BED SECTION OF THE DAM (Scale 1:1000)



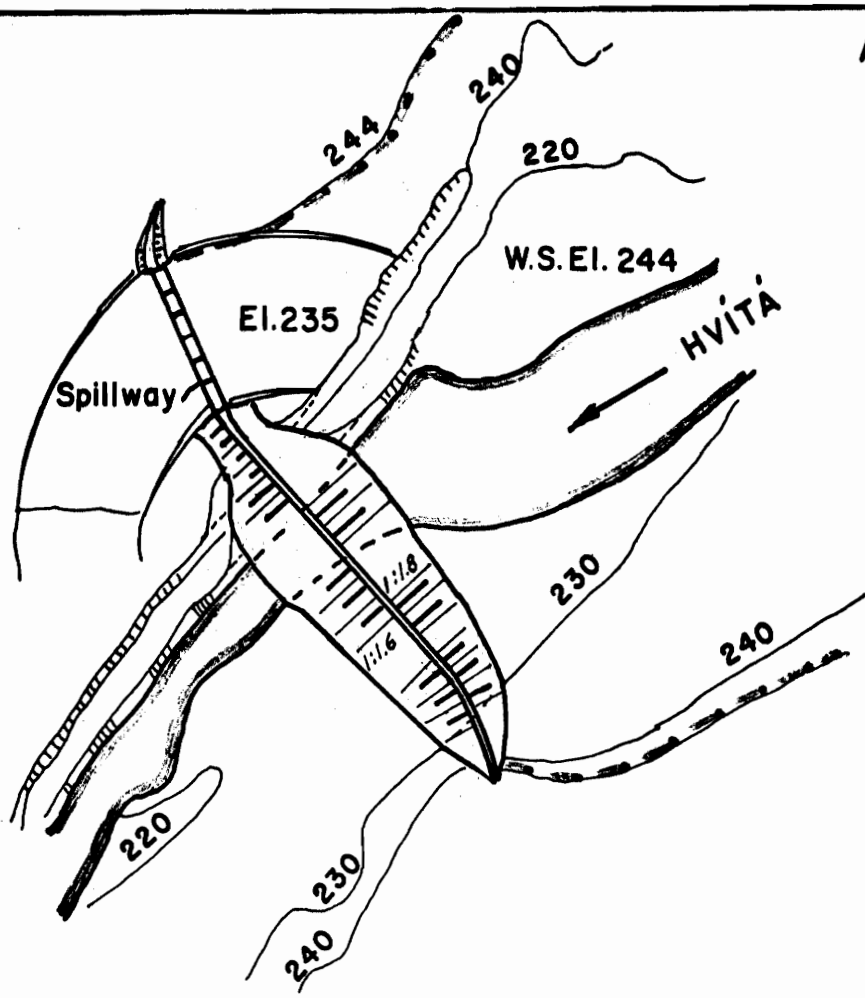
SECTION OF THE RIVERSIDE DAM (Scale 1:1000)



SPILLWAY SECTION (Scale 1:200)

HVÍTÁRVATN DAM
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND, REYKJAVÍK, ICELAND

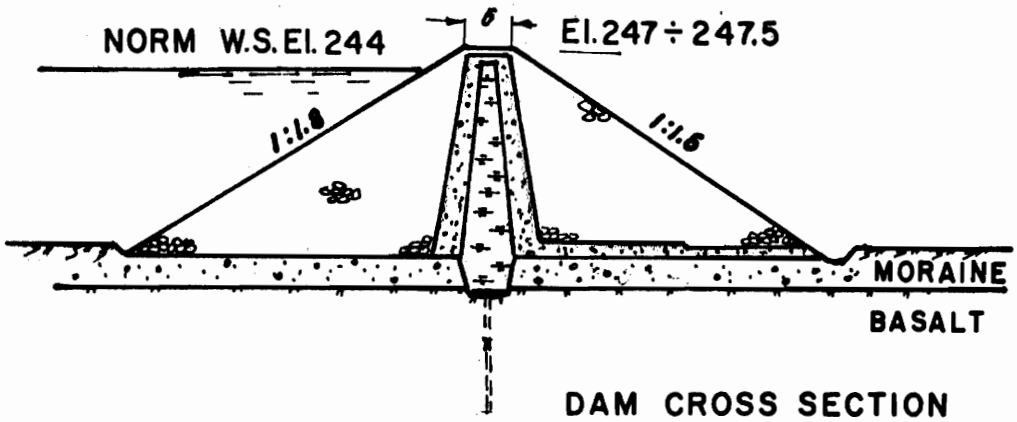
Appendix 4



GENERAL DAM PLAN

Scale 0 50 100 Meters

1:5.000

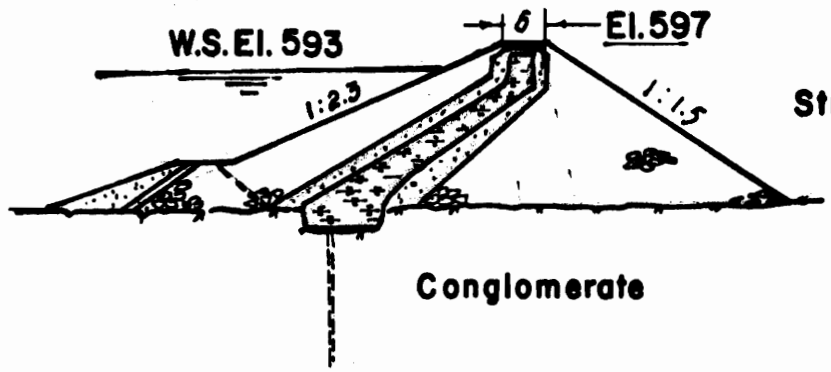


DAM CROSS SECTION

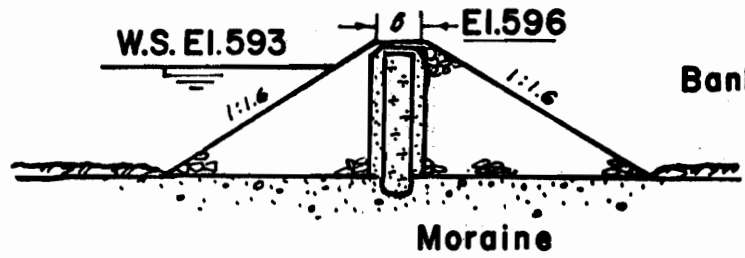
Scale 0 10 20 Meters

1:1000

GULLFOSS DAM
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND, REYKJAVÍK, ICELAND

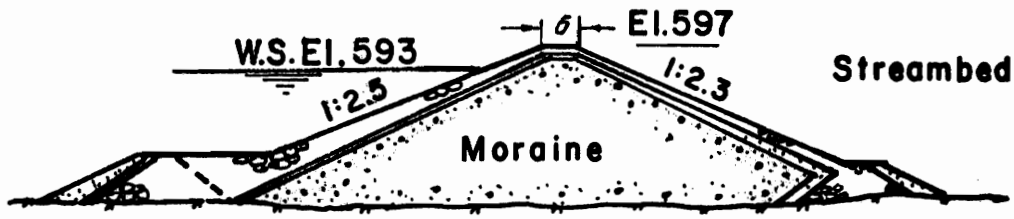


Streambed section



Bank section

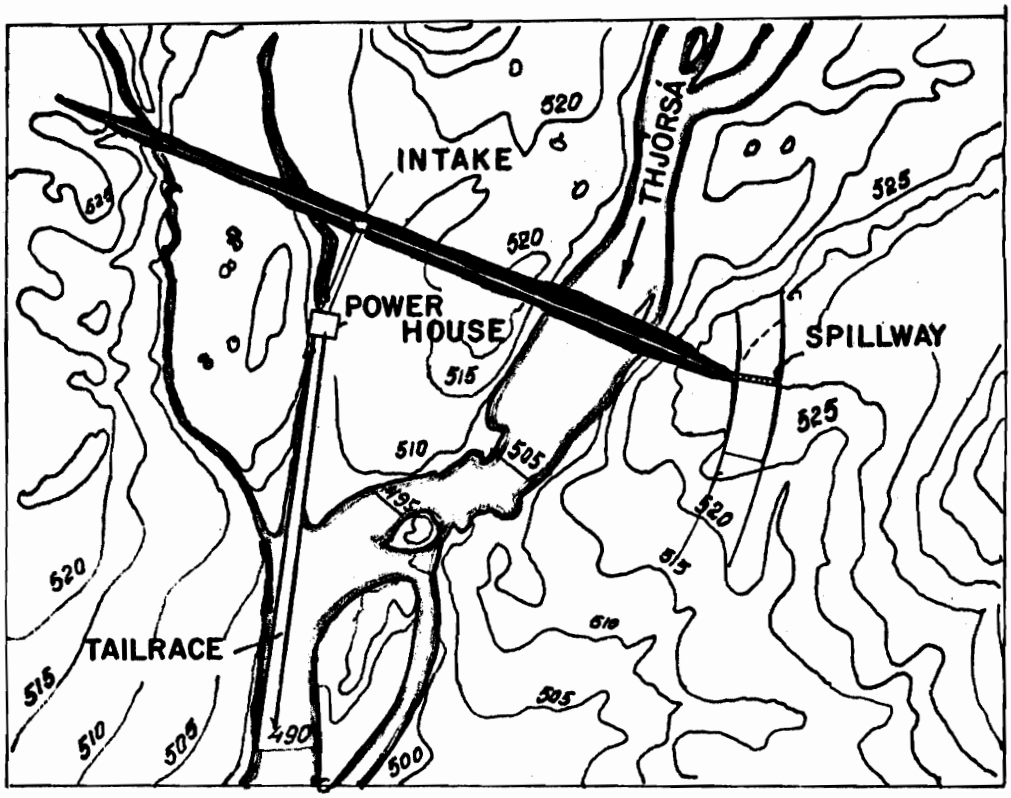
ROCK-FILL DAM



Streambed section

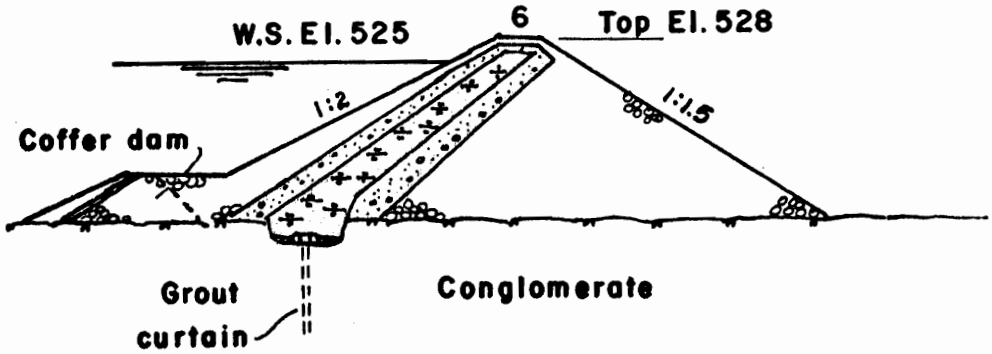
EARTH DAM (ALTERNATIVE)

NORDLINGAALDA DAM
THE STATE ELECTRICITY AUTHORITY, ICELAND
UNITED NATIONS SPECIAL FUND, REYKJAVIK, ICELAND



Scale 0 200 400 Meters

1:20000

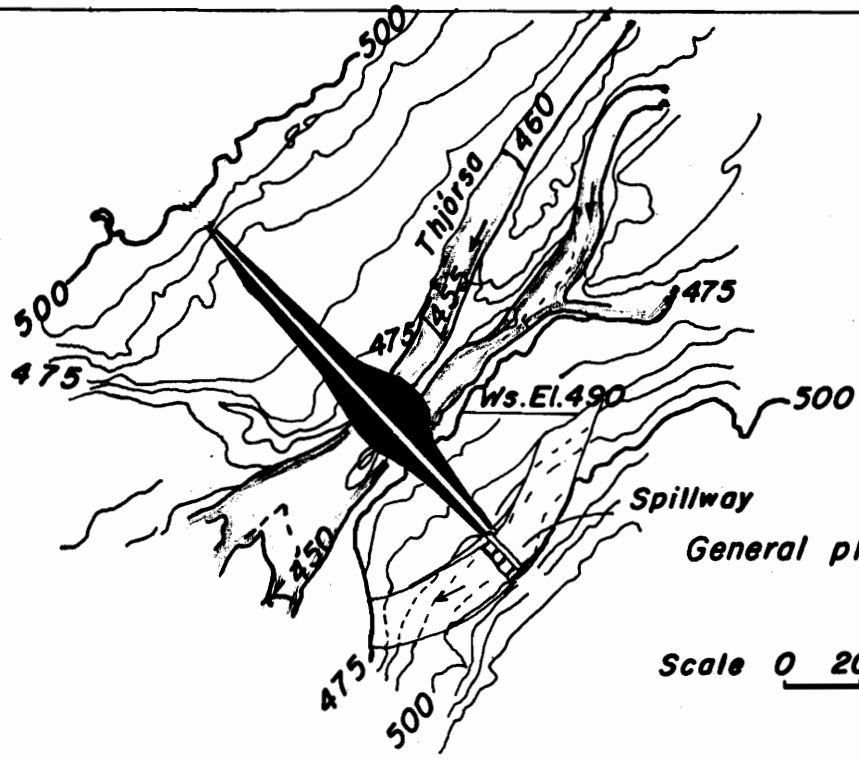


DAM CROSS SECTION

Scale 0 10 20 Meters

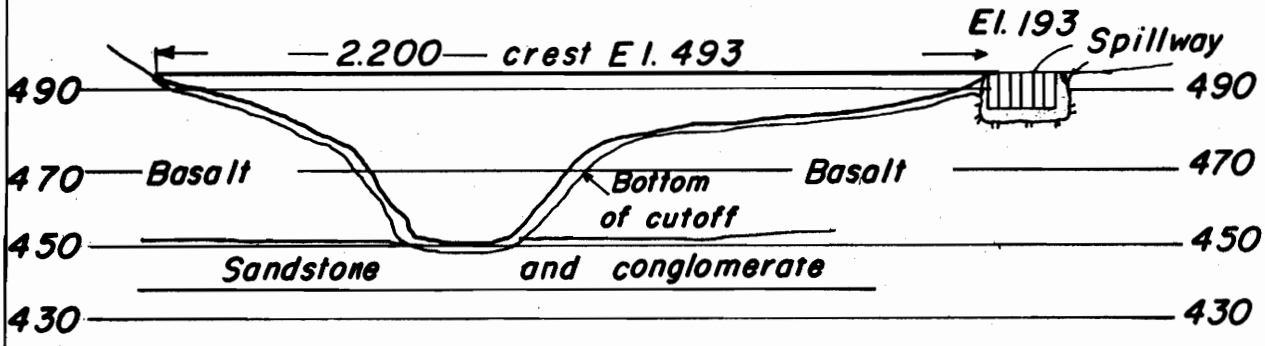
HVANNGILJAFOSS DAM
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND, REYKJAVÍK, ICELAND

Appendix 7



General plan

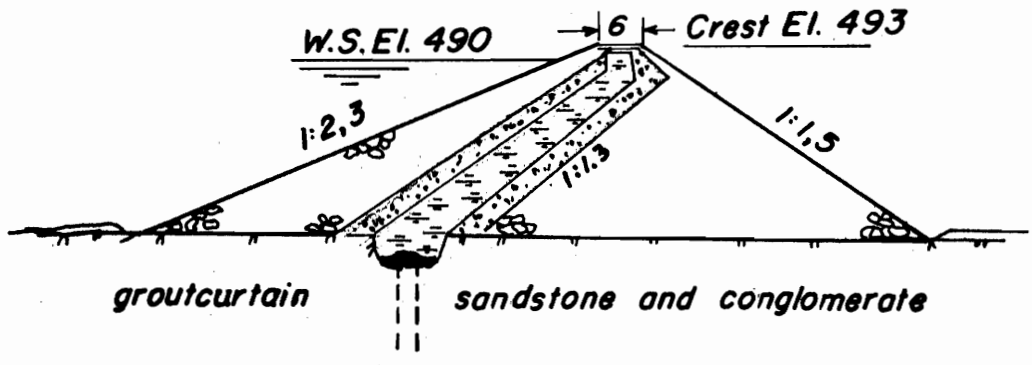
Scale 0 200 400 meters



Section along cut-off

Scale 0 100 200 meters

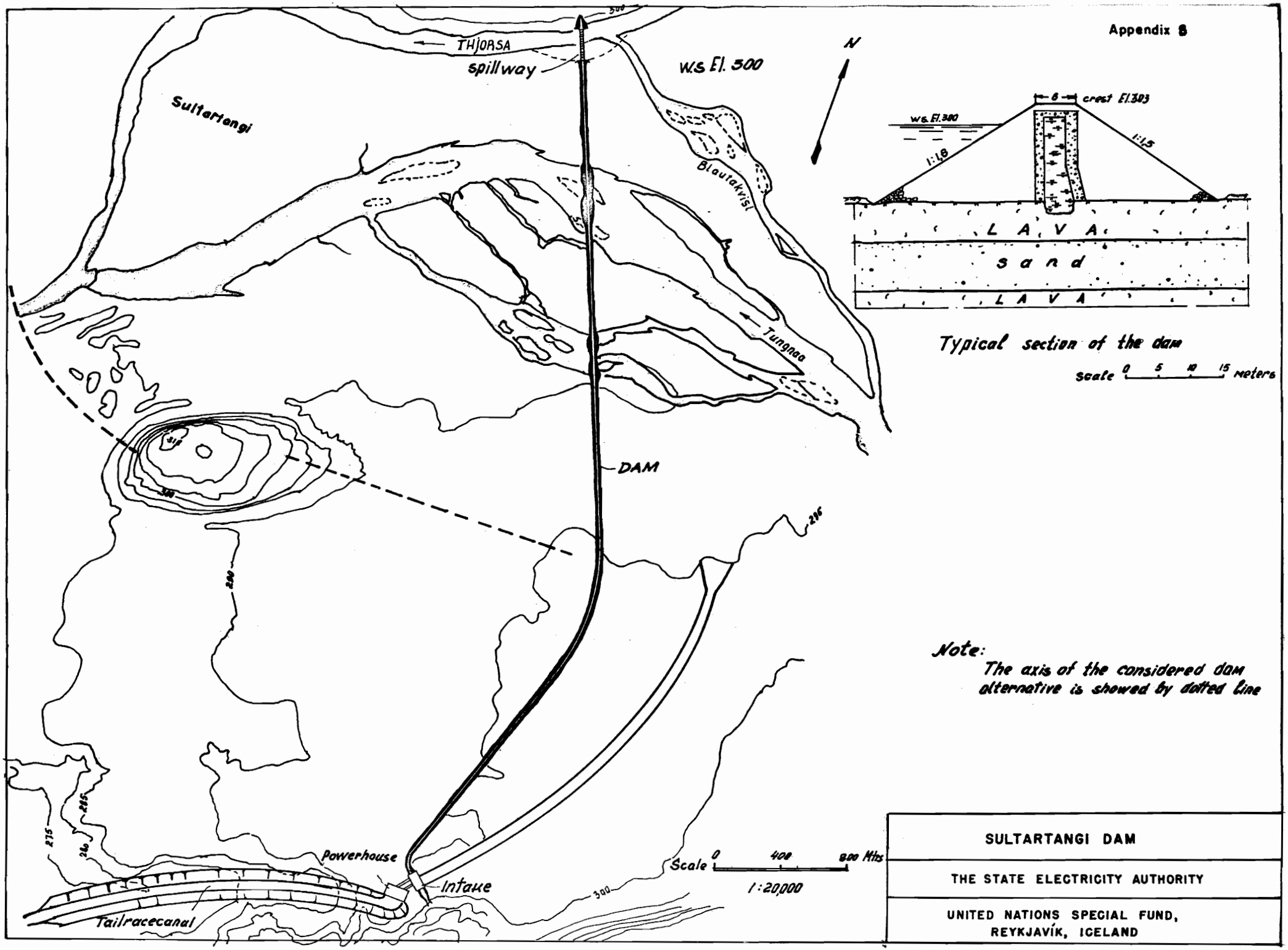
Horizontal scale



Typical Dam section

DYNKUR DAM
THE STATE ELECTRICITY AUTHORITY,
UNITED NATIONS SPECIAL FUND, REYKJAVIK, ICELAND

Appendix 8

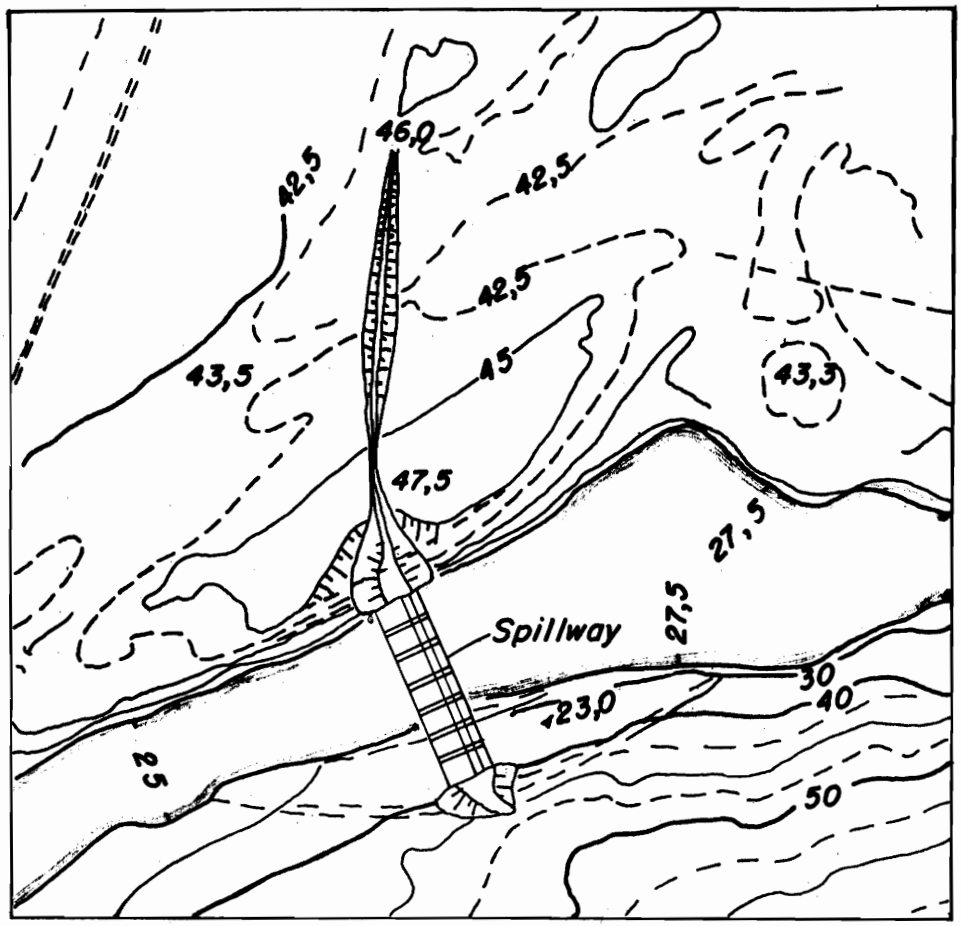


Typical section of the dam

Scale 0 5 10 15 meters

Note:
The axis of the considered dam alternative is showed by dotted line

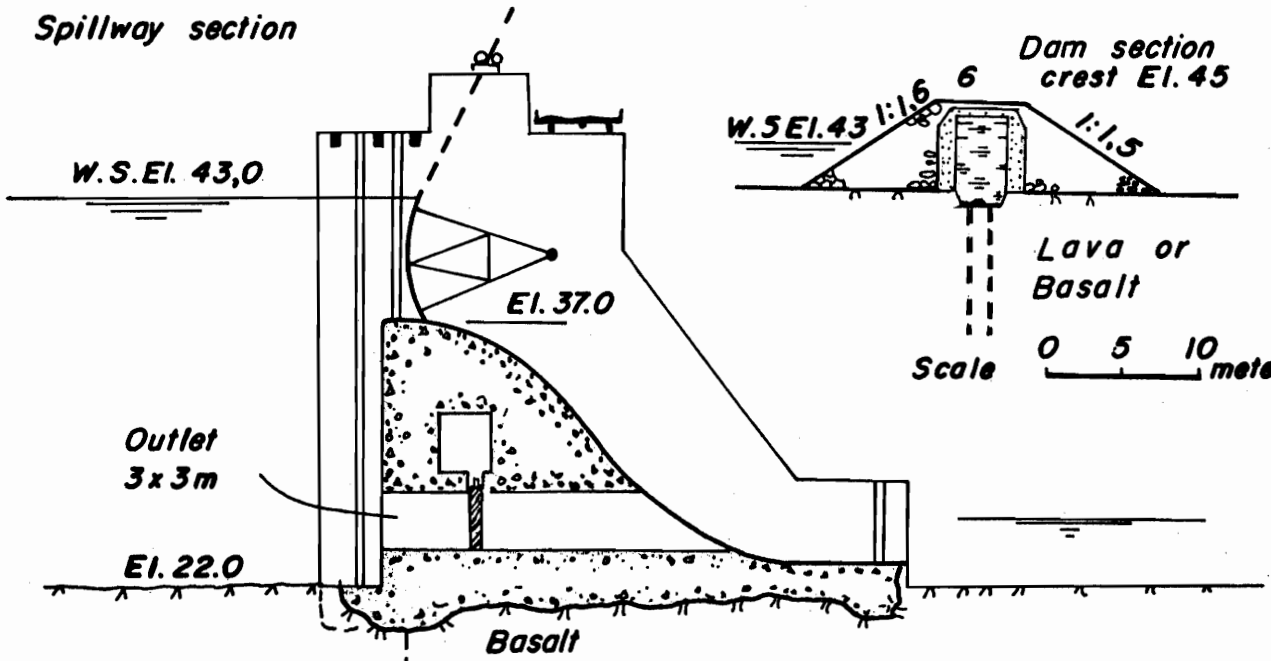
SULTARTANGI DAM
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND, REYKJAVÍK, ICELAND



Scale 0 50 100 meters

1 : 50.00

Spillway section



Scale 0 4 8 meters

Dam section crest El. 45

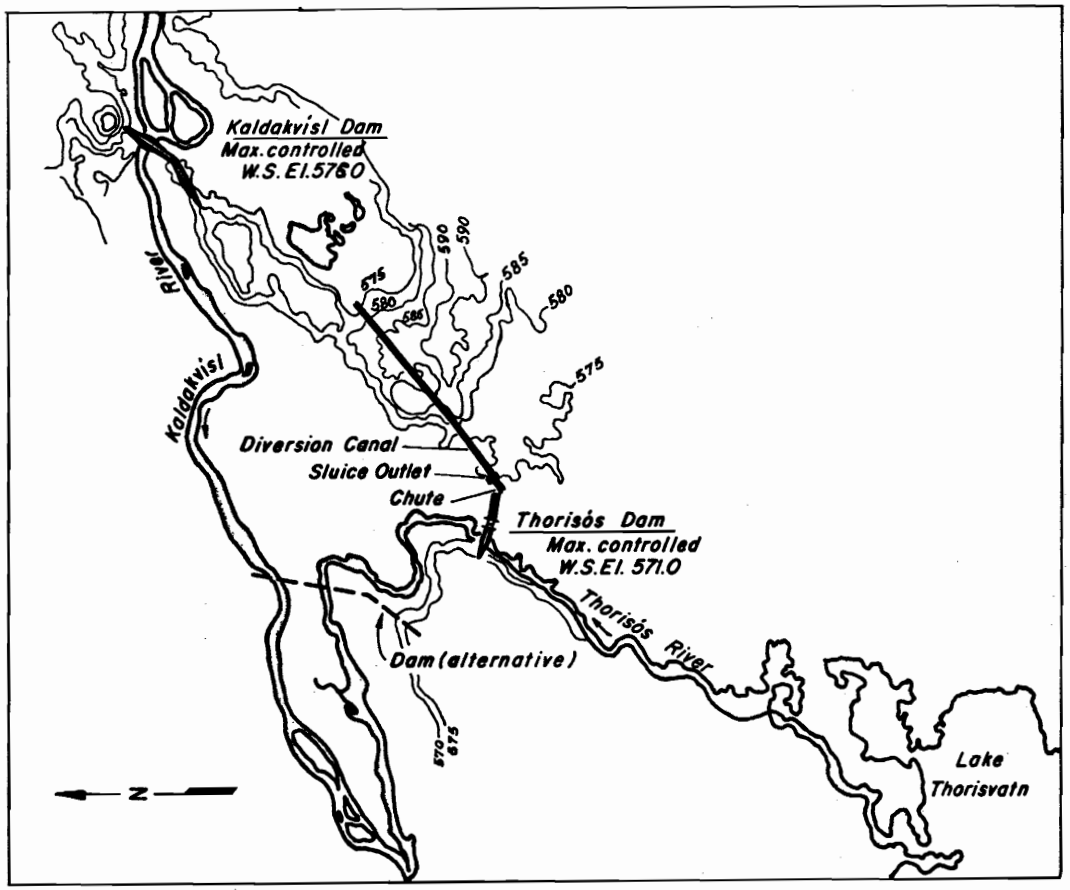
Lava or Basalt

Scale 0 5 10 meters

URRIDAFOSS PROJECT

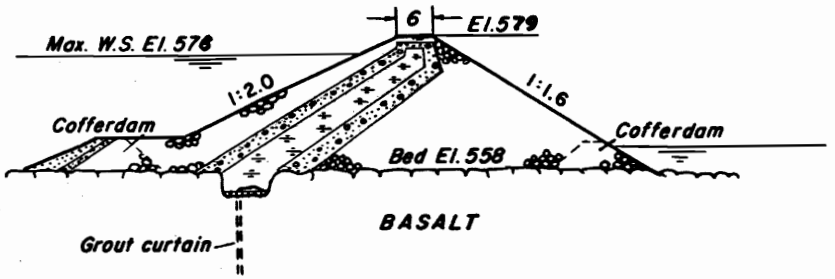
THE STATE ELECTRICITY AUTHORITY,

UNITED NATIONS SPECIAL FUND,
REYKJAVÍK, ICELAND

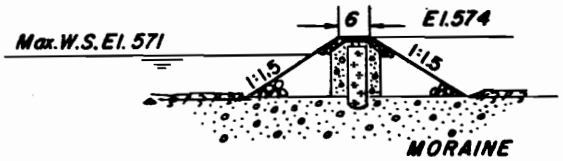


Scale 0 500 1000 1500 Meters
1:40.000

GENERAL PLAN

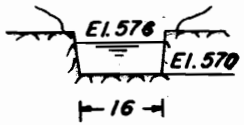


KALDAKVÍSL DAM (Streambed section)



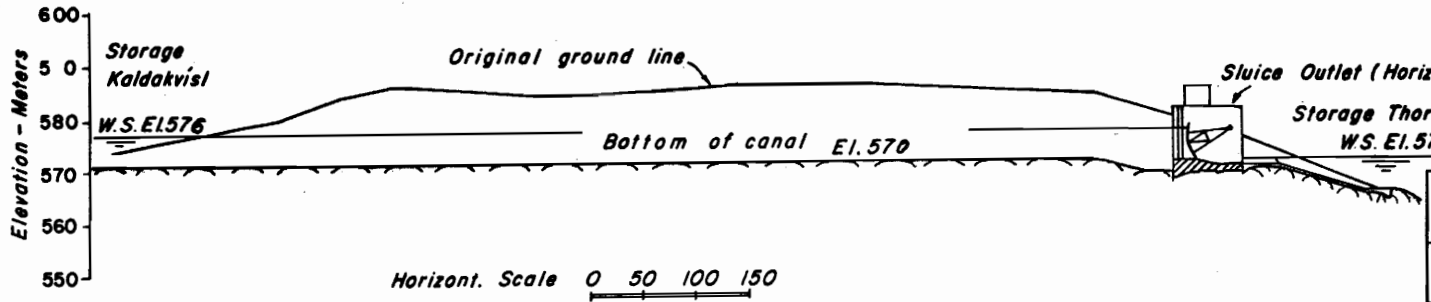
THORISÓS DAM

CANAL SECTION

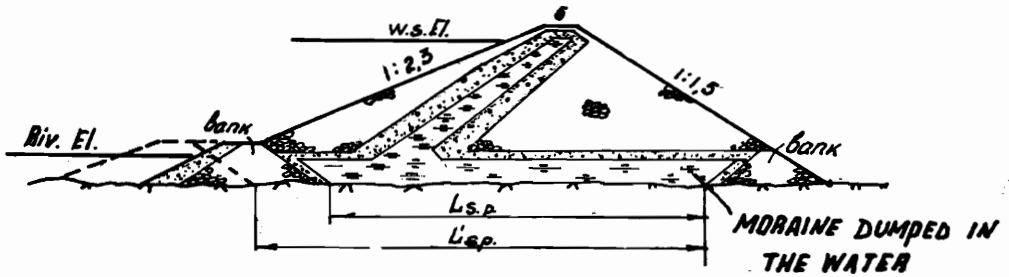


Note: The profile of the canal are drawn on the Basis of 3342 map

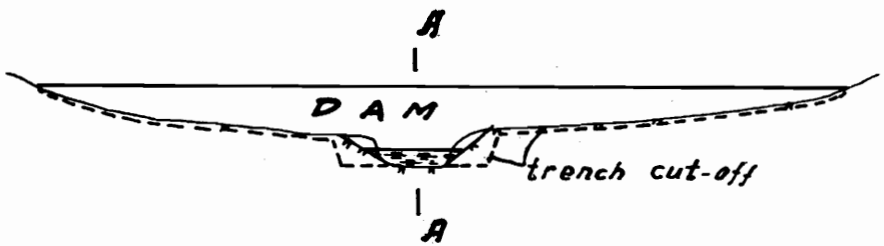
CANAL PROFILE



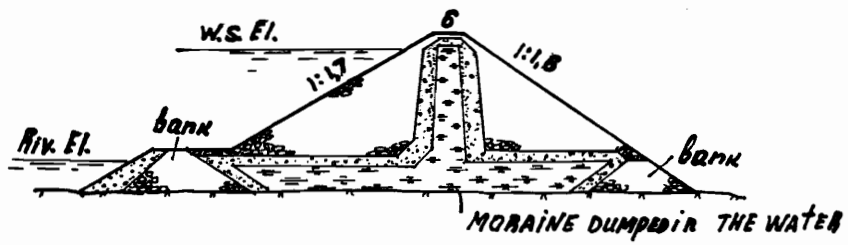
KALDAKVÍSL AND THORISÓS DAMS
THE STATE ELECTRICITY AUTHORITY,
UNITED NATIONS SPECIAL FUND, REYKJAVÍK, ICELAND



Section A-A (With sloping core)

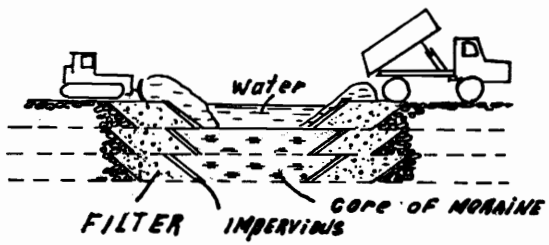


TYPICAL PROFILE OF THE DAM



Section A-A (with central core)

SCHEME DUMPED OF MORAINE IN THE WATER



SCHEME DUMPED OF MORAINE OF CENTRAL CORE

NOTE:

THE DRAWN DAM TYPES ALLOW NOT TO CARRY DOWN ANY GROUT CURTAINS

DAMS WITH IMPERVIOUS ELEMENTS OF MORAINE DUMPED IN WATER
THE STATE ELECTRICITY AUTHORITY
UNITED NATIONS SPECIAL FUND, REYKJAVIK, ICELAND

	NAME CHARACTERISTICS	HVÍTÁRVATN	GULLFOSS	NORDLINGAALDA	HVAMMGILJAFOSS	DYNJUR	SULTARTANGI
1	TYPE	Rockfill with sloping core	Rockfill with central core	Rockfill with sloping core	Rockfill with sloping core	Rockfill with sloping core	Rockfill with central core
2	TOP WIDTH m	6	6	6	6	6	6
3	LENGTH m	2.280	475	7.200	1.600	1.100	5.600
4	UPSTREAM slope	1:2,2	1:1,8	1:2,4	1:2	1:2,3	1:1,6
5	DOWNSTREAM slope	1:1,6	1:1,6	1:1,5	1:1,5	1:1,5	1:1,5
6	TOP ELEVATION m	443	247	597	528	493	303
7	RIVER BED EL.	414	213	557	506	453	288
8	ROCKFILL cm	1.056.000	201.000	2.248.000	566.000	886.250	1.052.000
9	FILLTER cm	301.000	61.000	665.000	160.000	172.000	185.000
10	CORE MATERIAL cm	187.000	43.000	524.000	104.000	171.000	185.000
11	CONCRETE cm	2.840	6.000	26.400	4.300	8.245	32.500
12	DESIGN FLOOD Kiloliters/Second	700	2.100	2.00	2.000	2.500	4.300
13	SPILLWAY SPECIFICATIONS.	6 gates 3x15	6 gates 5x15	6 gates 5x15	6 gates 5x15	7 gates 5x15	12 gates 5x15
14	FOUNDATION	moraine, basalt	moraine, basalt	conglomerate moraine	moraine	basalt	lava
15	COST in Million U.S.$	5,4	1,71	12,3	2,4	4,5	6,4

Notes:

1. The seventh line gives River level elevations since River level elevations since Riverbed elevations not available
2. Dam costs (15th line) do not include taxes and duties

HVÍTÁ AND THJÓRSÁ RIVER DAM DATA

THE STATE ELECTRICITY AUTHORITY

UNITED NATIONS, SPECIAL FUND, REYKJAVÍK, ICELAND

	NAME CHARACTERISTICS	SKARD	URRIDAFOSS	THORISVATN	TUNGNAÁRKRÓKUR	HRAUNEYJAFOSS	BÚDARHÁLS
1	TYPE	Rockfill with central core	Concrete dam and Rockfill central core.	Rockfill dams: Kaldak.-sl.core Th.ós-centr.c.	Concrete faced Rockfill dam	Concrete gravity dam	Rockfill with central core
2	TOP WIDTH m	6	6 for rockfill dam	6	6	4	6
3	LENGTH m	1.600	400	K.- 800 T.- 1010	900	1670	680
4	UPSTREAM slope	1:1,6	1:1,6	K.- 1:2 T.- 1:1,6	1:1,4	1:0,002	1:1,7
5	DOWNSTREAM slope	1:1,5	1:1,5	K.- 1:1,5 T.- 1:1,5	1:1,5	1:0,75	1:1,6
6	TOP ELEVATION	123	45	K.- 579 T.- 574	502	427,5	328
7	RIVER BED EL.	107	26	K.- 560 T.- 563	460	413	306
8	ROCKFILL cm	238.000	20.000	K.- 155.500 T.- 48.000	270.000	-	164.500
9	FILLTER cm	43.000	5.600	K.- 41.000 T.- 14.000	30.000	-	37.000
10	CORE MATERIAL cm	41.000	3.500	K.- 25.000 T.- 9.000	-	-	33.000
11	CONCRETE cm	27.300	41.000	K.- - T.- 5.000	face-8.000 spillw.-3.500	57.000	18.000
12	DESIGN FLOOD Kiloliters/Second	4.300	4.500	Total- 500	500	1.500	2.500
13	SPILLWAY SPECIFICATIONS	7-6x15	7-7x15	2-5x15	1-5x20	4-5x15	7-5x15
14	FOUNDATION	basalt	basalt, lava	K.-basalt T.-lava, palagonité	lava, móberg	lava, móberg	moraine, basalt lava
15	COST in million U.S.$	2,26	1,6	Total- 1,75	1,83	2,6	1,7

Notes:

1. The seventh line gives River level elevations since Riverbed elevations not available
2. Dam costs (15th line) do not include taxes and duties

THJÓRSÁ AND TUNGNAÁ RIVER DAMS DATA

THE STATE ELECTRICITY AUTHORITY

UNITED NATIONS, SPECIAL FUND
REYKJAVÍK, ICELAND