FOUNDATION TREATMENTS OF FILL DAMS IN JAPAN

Committee on Rock Mechanics J.S.C.E.

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Foundation Treatments of Fill Dams in Japan

CONTENTS

Part 1 Geological Features

1. General

Dam foundation is required to be mechanically stable for the dam structure and hydraulically stable against seepage flow. In case of fill dam, for the reasons that the area of the dam basement is so wide as to reduce stresses and that the dam body can follow the deformation of the foundation to some extent, the mechanical condition required for the foundation of a fill dam is more allowable than that of a concrete dam. However, since especially the stability against seepage through the foundation around the contact with dam body affects directly the stability of the core of fill dam, careful examination, proper design and treatment are required.

The foundation rocks have various geological properties respectively, such as rocks with many joints though they are hard, rocks with distinct faults, extremely weathered rocks, and so on. In relation to these properties, the foundation causes different engineering problems in design and construction of dam, and in the foundation treatment.

This part of the report summarizes the rock properties of the foundation of existing fill dams in Japan, concerning the kinds of rocks, and describes the outline of engineering problems, investigations and tests, and treatment methods of foundation in reference to rock properties, based on the statistically analysed results.

2. Kinds and properties of foundation rocks

(1) Classification of rock kinds and properties

The kinds of fill dam foundation rocks are classified into the following 10 categories of rocks based on the existing dam cases.

a. Sedimentary rocks of Palaeozoic to Mesozoic
b. Crystalline schists
c. Gneisses
d. Granites
e. Acid volcanic rocks of late Mesozoic to Palaeogene
f. Sedimentary rocks of late Mesozoic to Palaeogene
g_1. Sedimentary rocks of Neogene
g_2. Deposits of Quaternary
h_1. Volcanic rocks of Neogene
h_2. Volcanic rocks of Quaternary

The foundations of 44 dams studied this time can be classified as shown in the following Table-1

Table 1 Numbers of fill dams in reference to the kinds of rocks

Kinds of rocks	a	b	c	d	e	f	g_1	g_2	h_1	h_2	Total
Nos. of dam cases	9	4	2	7	5	3	6	3	4	1	44

in reference to the kinds of the rocks distributed in the river beds.

On the other hand, rock properties are classified into the following 5 types.

1) Rocks with many joints, rocks with open cracks
2) Rocks with distinct faults
3) Extremely weathered or altered rocks
4) Soft rocks
5) Unconsolidated deposits

The following analyses are made based on the above classifications.

(2) Topographical and geological features of the respective kinds of rocks

a) Topographical features

Fig. 1 shows the general forms of valleys at dam sites by the ratio (H/L) of dam height (H) to dam crest length (L) in reference to the kinds of rocks distributed on the river beds. According to the diagram, though the values are dispersed, there are approximately the following trends in reference to the kinds of rocks.

- o In the sedimentary rocks of Cenozoic, there are many of about 1/6 in H/L, showing that the valleys are wider than those of the other kinds of rocks.
- o Granites are divided into two groups of H/L = 1/3 to 1/4 (5 cases) and H/L = 1/2 to 1/3 (2 cases). In every case of the former, the intensely weathered zone is thick in the portion above the middle of the slope.
- o In the acid volcanic rocks of the late Mesozoic to Palaeogene, the ratio is almost about 1/3.

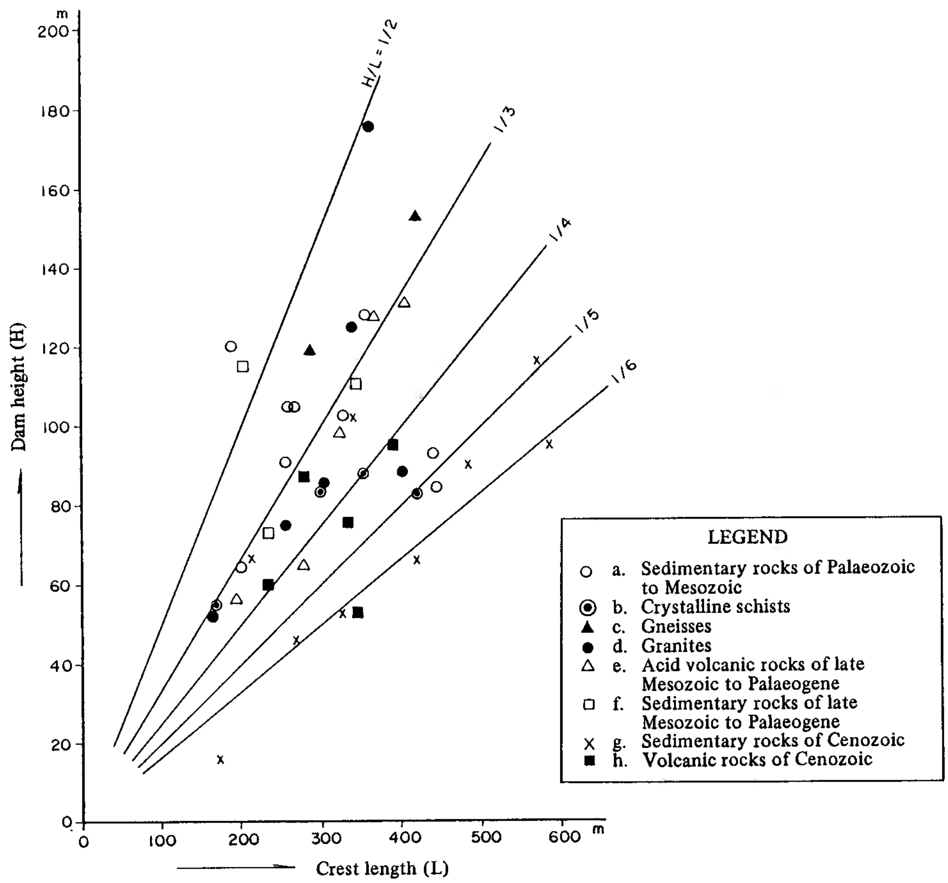


Fig. 1 Relation between dam height and dam crest length

- In case of the sedimentary rocks of Palaeozoic to Mesozoic and the sedimentary rocks of the late Mesozoic to Palaeogene, the values are generally dispersed, but there are many of more than 1/3 in H/L. Of the former, the two cases of H/L = about 1/5 are in the Hidaka Group of Mesozoic.

b) Geological features

Geological features in reference to the kinds of rocks of dam sites are collectively shown in Table 2. The kinds of rocks are not limited to those distributed on the river beds only, but include those distributed in the entire dam sites (partially including ridges). Therefore it happens that one dam site includes more than one kind of rocks. Furthermore it similarly happens that one kind of rocks includes different geological features.

According to Table 2, there are certain trends of geological features in the respective kinds of rocks. The contents can be summarized as follows:

(i) Sedimentary rocks of Palaeozoic to Mesozoic;

Many faults are developed. The width is less than 1.5 m in many cases, but there is a case with about 15 m width. In some cases, open joints are developed by creeping on a slope. When intrusive rocks occur, there are altered zones and fracture zones around them in some cases, and open joints are developed in the intrusive rocks themselves.

(ii) Crystalline schists;

There are many faults, and those running along the schistosity are large in scale. There is a case with 30 m width. In many cases, the slope is often weathered and intensely jointed deep inside by creeping on the slope occurring along the faults.

(iii) Gneisses;

In the portion above the middle of the slope, the intensely weathered zone is thick. Some faults are large in scale. Joints and seams are developed very deep inside in some cases.

(iv) Granites;

On a gentle slope or on a slender ridge, the intensely weathered zone is thick. Some faults are large in scale.

(v) Acid volcanic rocks of late Mesozoic to Palaeogene;

Large-scale faults are developed especially near the river bed, and the river bed is wide topographically in many cases.

(vi) Sedimentary rocks of late Mesozoic to Palaeogene;

Slates have open joints caused by faults and creeping, and have weathered zone deep inside in many cases.

(vii) Sedimentary rocks of Neogene;

Mudstones and some tuffs are soft. In one case, tuffs have large scale faults.

(viii) Quaternary deposits;

In general, deposits are not or little consolidated. Pyroclastic deposits often fills the ancient valley in many cases.

(ix) Volcanic rocks of Neogene;

Columnar joints are developed. Altered zones are formed around intrusive rocks in some cases.

(x) Volcanic rocks of Quaternary;

Remarkable open joints are developed. The ancient valley is filled with the volcanic products in many cases.

(3) Trends of dam construction with the lapse of years, and trends of dam height in the respective kinds of rocks.

Fig. 2 shows the relation between the kinds of rocks of dam foundation at river bed and the start year of impounding (including the planned year). As for dams founded on the sedimentary rocks (g_1, g_2) and volcanic rocks (h_1, h_2) of Cenozoic, 4 dams were constructed before 1974, in which 3 dams are of facing type. Core type was adopted in many dams constructed after 1975 (9 cases). These kinds of rocks have many geological problems such as soft rocks, unconsolidated deposits, waste-filled valley and open joints as mentioned before. And they have never been accepted as foundation rocks for large dams intentionally until recent years. The reasons why large dams have come to be constructed in such geological condition recently are surmarised to be that dam sites with good foundation rocks have been difficult to find out and that construction techniques, especially grouting technique has been much improved. According to the diagram, all the four cases in crystalline schists were also constructed after 1975.

Fig. 3 shows the dam height in relation to the kinds of rocks at river bed. According to it, on Quaternary deposits (g_2) and volcanic rocks of Neogene to Quaternary (h_1, h_2), the dam height is low, compared with those on other rocks, and there has been no case of a fill dam with a height of 100 m or more. One reason is summarized to have been the geological factor mentioned above. Also in crystalline schist areas (b), all the dams are less than 100 m in height.

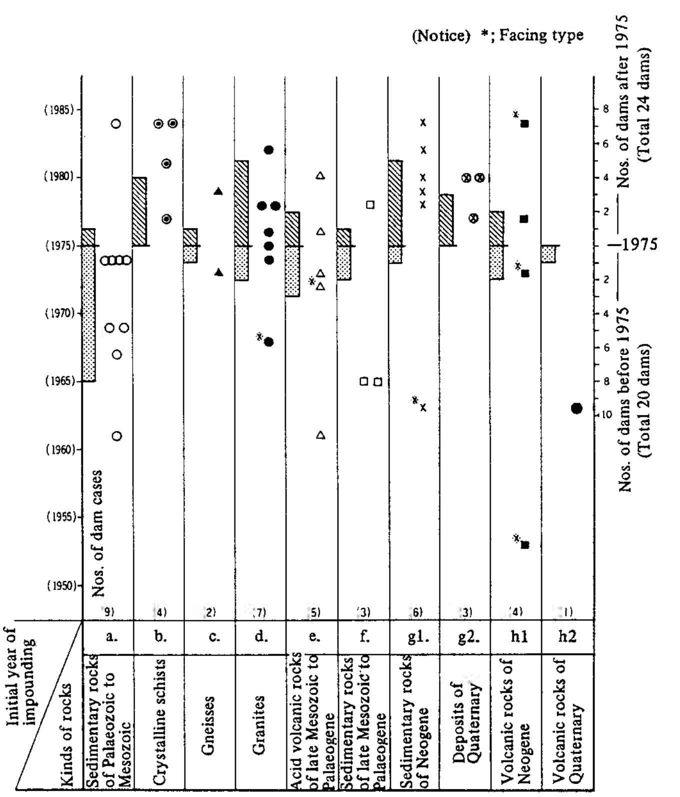


Fig. 2 Kinds of rocks at river bed and initial impounding years

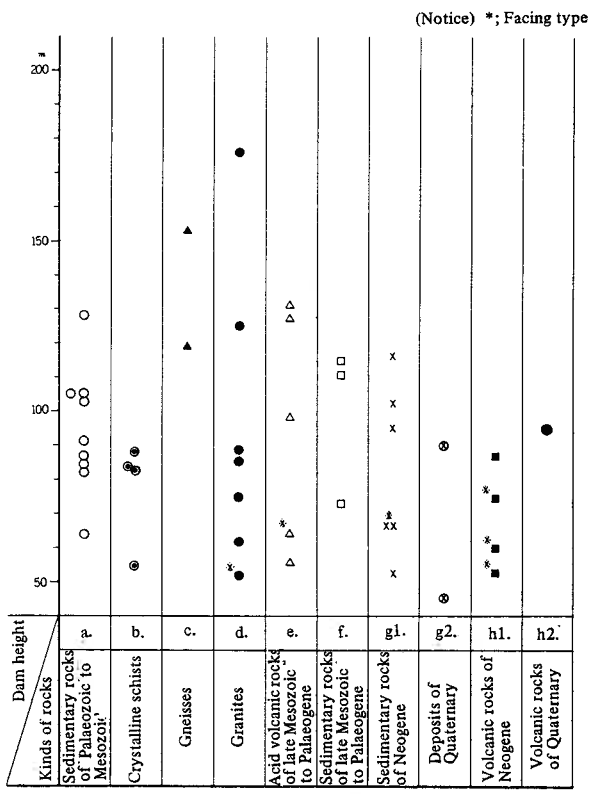
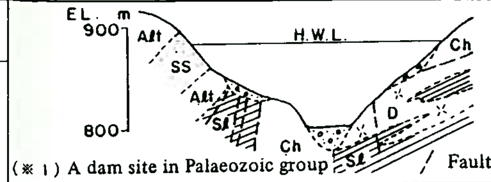
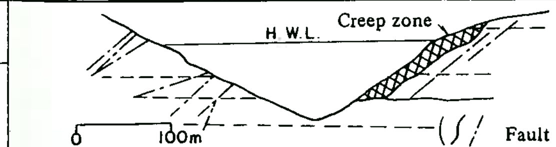
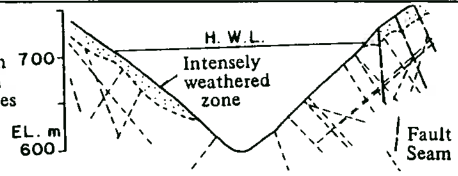
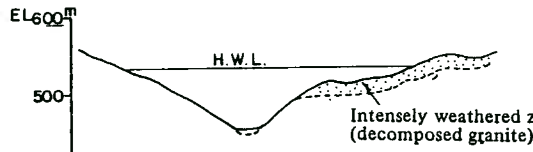
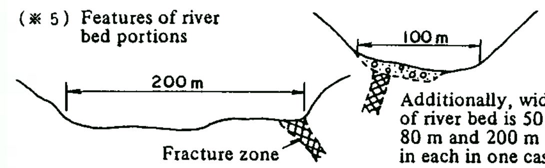
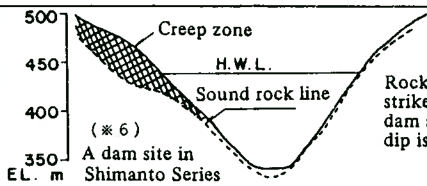
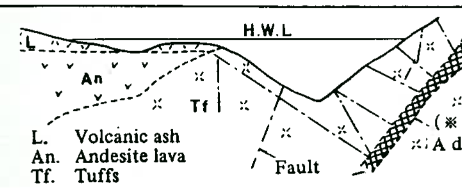
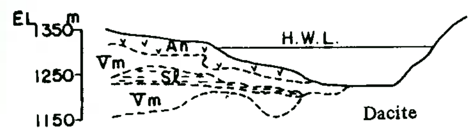
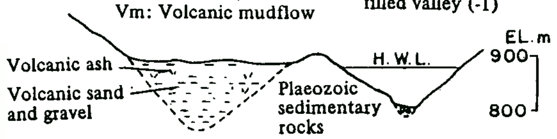
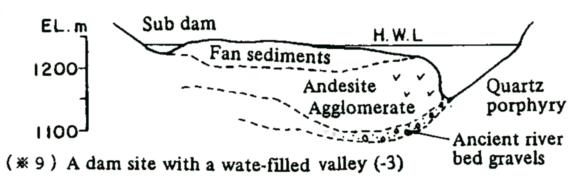


Fig. 3 Kinds of rocks at river bed and dam heights

Table 2 Geological features of fill dam foundations

└1 Nos. of dam cases where applicable rock properties occur.
└2 Proportion of corresponding dams to total nos. in the kind of rock.

Kinds of rocks	Nos. of dam cases	District or typical formation	Rock properties: ① Rocks with many joints, rocks with open cracks	② Rocks with distinct faults	③ Extremely weathered or altered rocks	④ Soft rocks	⑤ Unconsolidated deposits (Impervious core zone, including deep portion of foundation)	⑤ Unconsolidated deposits (Rock zone)	Examples of typical geologic section of dam site
a) Sedimentary rocks of Palaeozoic to Mesozoic	10	Hidaka Group of Mesozoic in Hokkaido district: 3 cases Chichibu System in Chubu, Hokuriku, Kinki and Kyushu districts: 5 cases Mesozoic formations in Hokuriku district: 2 cases	6 (50 100) o Open cracks in Mesozoic group caused by creeping on slopes (2 cases), in particular, width of which is 20 cm to 30 cm. o Distinct open cracks are developed in the intrusive rocks occurred in the sedimentary rocks.	8 (50 100) o Many faults are developed in Chichibu System, but the width is less than 1.5 m in 4 cases. In particular case, a fault with 15 m width is developed on a ridge. (*1) o In Hidaka Group, faults with 3 m width are distributed in one case, and many faults are developed around the intrusive rocks.	2 (50 100) o Hydrothermally altered zones are developed within and around the instrusive rocks in one case, the width of which is from 20 m to 40 m. o The upper portion from the middle of the slopes on both banks are weathered intensely to the deep in one case.	(50 100)	(50 100)	% o Nos. of dams where unconsolidated deposits were left on the foundation.—(4)	 (※1) A dam site in Palaeozoic group
b) Crystalline schists	4	Sanbagawa metamorphic belt: 2 cases Sangun metamorphic belt: 1 case	1 o Open cracks are developed by creeping of rocks overlying a fault.	4 o The strikes of faults not always correspond to the schistosity. The width of a fault running paralle to the schistosity is 30 m in one case, and that across the schistosity is less than 5 m in 4 cases.	2 o Rocks overlying faults are deeply weathered, caused by creeping on a slope in 2 cases. (*2)				 (※2) A dam site in crystalline schists (schistocity is almost horizontal)
c) Gneisses	2	Hokuriku district	1 o Joints and seams are developed to the deep portion of the slope. (*3)	1 o A fault with 20 m width is running on a slope, dipping toward the hillside in one case.	2 o Although the slope is steep with the inclination from 40° to 45°, the intensely weathered zone is 20 m thick in the portion above the middle of the slope in 2 cases; in every case, the rock is biotite gneiss.			(1)	(※3) A dam site in gneisses 
d) Granites	7	Tohoku district: 2 cases Chubu district: 3 cases Chugoku district: 2 cases	2 o Fine joints are generally developed. o Open joints with several mm width are developed to the depth of 30 meters on the slope with inclination over 45°	2 o Faults are generally large in scale, such as with 7 m width in one case, and some with width from 5 m to 14 m in another case.	5 o Intensely weathered zone is thick on a slope in cases. In one case among them, hickness reaches from 30 m to 40 m near the crest of dam. On a gentle slope or a slender ridge, weathered zone is generally thick. (*4)			(3)	 (※4) A dam site in granites
e) Acid volcanic rocks of late Mesozoic to Palaeogene	6	Chubu or Kinki district	1	5 o Near the river bed, some faults with width from several meters to 15 m are developed in 3 cases, and that from 30 m to 50 m in one case. o A fault with 15 m width is running across a ridge in one case. o River bed is generally wide, probably because of large scale faults (*5)	1 o On a gentle slope, intensely weathered zone is rather thick.			(2)	(※5) Features of river bed portions 
f) Sedimentary rocks of late Mesozoic to Palaeogene	3	Shimanto Series in Chubu, Kinki and Shikoku districts	3 o Open cracks, often containing flow-in clyas, are developed to the deep portion, caused by faults and creeping on a slope.	2 Faults, both that parallel to and that across the strike of strata, are developed. The width is almost less than one meter, but a few are with width from 5 m to 10 m.	2 o In the slate-rich zone, the rocks are intensely weathered to the deep portion, caused by faults and creeping on a slope. There is a case where horizontal depth of weathered zone reaches about 50 m near the crest of dam (*6)				 (※6) A dam site in Shimanto Series
g1) Sedimentary rocks of Neogene	8	Northern area from Kanto district	2 o Distinct open cracks with width from 0.5 to 1 cm are developed in tuffaceous rocks in 2 cases.	3 o Each case is in tuffs. One case is with width from 20 m to 30 m containing clays with 5 m thickness, and another case is with 10 m width. (*7)	1 o In tuffs overlying faults.	4 o In mudstones, 2 cases. o In tuffs, 2 cases.		(2)	 (※7) A dam site in tuffs
g2) Deposits of Quaternary	8	Northern area from Kanto district	1 o Columnar joints are developed in welded tuffs.			1 o Unconsolidated pumice tuff	7 o Pyroclastic sediments fill up the ancient valley in 3 cases. (*8) In these cases, ancient river bed gravels are often deposited on the unconformity of foundation rocks. o Additionally, 1 case of terrace gravels, 1 case of alluvial fan sediments, 1 case of volcanic ashes and 1 case of existing river bed gravels.		 (※8-1) A dam site with a waste-filled valley (-1)
h1) Volcanic rocks of Neogene	5	Northern area from Kanto district	4 o Columnar joints are developed in rhyolites and andesites.	1 o Small scale faults are developed in rhyolites	1 o Hydrothermally altered zones with about 10 m width are developed at the contact between dacites and intrusive rhyolites.			(3)	 (※8-2) A dam site with a waste-filled valley (-2)
h2) Volcanic rocks of Quaternary	2	Kanto district: 1 case, Chubu district: 1 case	2 (50 100 %) o Distinct open cracks are developed in 2 cases.	(50 100)	(50 100)	(50 100)	2 (50 100) o Ancient valleys are filled up with lavas in 2 cases, and ancient colluvials or river bed gravels are deposited on the unconformity of the foundation rocks. In this case, the surfaces of the Quaternary volcanic rocks make gentle slopes, as in the case of (g2).		 (※9) A dam site with a wate-filled valley (-3)

3. Rock properties and engineering problems as fill dam foundations

Fig. 4 is a histogram showing, based on the results of the questionnairing, what problems were in particular caused in dam design and construction by the respective rock properties.

Engineering problems \ Rock properties (Nos. of dam cases)	① Rocks with many joints, rocks with open cracks	② Rocks with distinct faults	③ Extremely weathered or altered rocks	④ Soft rocks	⑤ Unconsolidated deposits: Core zone *	⑤ Unconsolidated deposits: Rock zone
	23	26	16	5	9	15
Mechanical stability	2	13	12	1	6	15
Stability against seepage	23	19	11	4	9	
Stability of slopes		3	1			
Execution of grouting	8	6	10	4	6	
Low groundwater level in the ridge	4		1			
Slaking of foundation rocks		1		1		

(Notice) * ; Including deep portion of the foundation.

Fig. 4 Rock properties and engineering problems (including multiple counting)

(1) Rocks with many joints, rocks with open cracks;

Since these rocks are generally very high in permeability, careful study was especially made on seepage. Especially in the lava of Quaternary, remarkable open joints are developed. Furthermore in crystalline schists and the sedimentary rocks of pre-Neogene, open joints are developed deep inside by creeping on a slope in many cases.

For execution of grouting, special consideration was made when the opening of crack was large and when joints are dense to make a fracture zone.

Since the ground water level in the ridge is generally low in the rocks with these rock properties, curtain grouting (rim grouting) was executed in a very wide range, in 4 cases.

(2) Rocks with distinct faults;

Out of 26 cases in which this property was especially examined, 19 cases (80%) took up the problems on stability against seepage along faults and their peripheral fracture zones. With regard to mechanical stability, the deformation of the foundation was examined in 13 cases (50%) where the faults were relatively wide. When the faults and their surrounding rocks were soft, carefull consideration was made on the grouting pressure and grout materials in many cases. In a case where grouting work was planned deep inside a rock mass along faults, grout tunnels or grout shafts were arranged for the convenience of the effective grouting work.

(3) Extremely weathered or altered rocks;

In 12 cases (about 75%) out of 16 cases, the bearing capacity of the foundation was carefully examined, and, as the result, the portion with insufficient bearing capacity was removed by excavation, and, then careful grouting was made to the remaining rocks in order to secure sufficient watertightness. However, since weathered rocks and altered rocks are generally permeable and soft, prudent consideration was paid on the grouting method and materials in many cases.

(4) Soft rocks;

In case of soft rocks, seepage problem, especially piping action was carefully studied in 4 dams out of corresponding 5 dams. Since the breaking strength of the foundation against grouting pressure[*1)] is generally low, grouting pressure and speed were carefully controlled at the execution of grouting. As for mechanical stability, on the other hand, no large-scale treatment was encountered in any case, though common investigation was made on the foundation. In the case shown in Fig. 4, the bearing capacity was examined, considering that the excavated surface of mudstone might be deteriorated by slaking.

(5) Unconsolidated deposits;

a) Foundation of impervious core zone (1. contact portion with dam body)

Unconsolidated deposits were left at the direct foundation of impervious core zone in 6 dams in Table 2. These cases, in reference to geology, include 2 cases of volcanic mud flow sediments, 1 case of alluvial fan sediments, 1 case of terrace gravels, 1 case of existing river bed gravels and 1 case of volcanic ashes. In all of these cases, the problem in seepage, especially piping was carefully examined, and mechanical stability such as bearing capacity and deformation of foundation was also examined. Since these sediments are mostly unconsolidated and highly permeable, effectiveness of grouting often comes into question.

In case of volcanic mud flow sediments, the matrix is generally slightly permeable. However, when the sediments contain boulders and/or volcanic sand as lumps, the sediments be highly permeable, and are heterogeneous in view of permeability on the whole.

b) Foundation of impervious core zone (2. deep portion inside the foundation)

In 5 cases, the ancient valley had been filled with pyroclastic rocks and lavas of Quaternary. In these cases, an unconsolidated stratum of old river bed sediments overlies unconformably the foundation rocks. With regard to the geology of sediments, the cases include 2 cases of ancient river bed gravels, 1 case of colluvials, 1 case of lacustrine sediments, and 1 case of volcanic ashes and scoria beds. Since these sediments are generally unconsolidated and highly permeable, especially careful consideration is required against seepage as mentioned in case of a).

For a waste-filled valley, considerable investigation was commonly carried out to know the distribution and the heterogeneous properties of sediments, and the situation of unconformity.

c) Foundation of rock zone

In 15 cases, unconsolidated deposits were left on the foundation for the rock zone of dam. In

*1) "breaking strength of the foundation against grouting pressure" is to be called as "critical pressure of the foundation" after this.

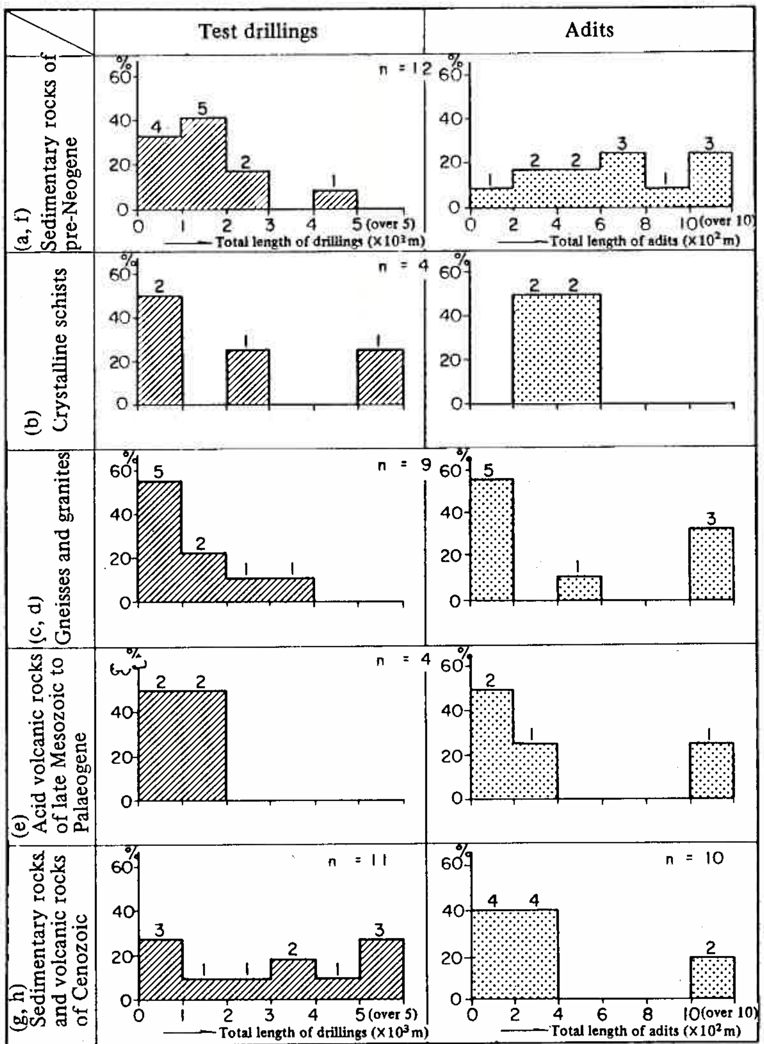
these cases, mechanical stability was sufficiently examined on bearing and deformation capacity of the gravel bed. The unconsolidated layer left was generally of river bed gravels or terrace gravels. Special cases include 3 cases of talus deposits, 1 case of lacustrine sediments, 1 case of alluvial fan sediments and 1 case of colluvial deposits.

4. Investigations and tests

With regard to the contents and the quantity of investigations and tests, comparison was made in reference to kinds and properties of rocks and dam scales, but no remarkable trend was identified on the whole.

(1) Trend in rock properties

According to Fig. 5, in the sedimentary rocks and volcanic rocks of Cenozoic, adits were relatively short and almost same in length in every site, but test drillings were variable in depth and extremely long drillholes are conspicuously seen. On the contrary, in the sedimentary rocks of pre-Neogene, adits were



(Notice) a, b ... ; refer to chapter 1.
Number shows dam cases.

Fig. 5 Quantity of investigations for dam foundation in relation with the kinds of rocks

long and drillholes were relatively short.

The reason for the above is summarized to be that, in the former, drilling investigation can be applied effectively to know the almost horizontal structures of the strata, and that in the latter, on the contrary, long and many adits are required to confirm the faults and weathered zones.

(2) Relation with dam scale

According to Fig. 6, the relation between dam crest length and total drilling length is generally dispersed, but in much of the kinds of rocks positive relation can be observed. In case of the sedimentary rocks of pre-Neogene, no correlation is observed at all.

According to Fig. 7, between dam height and adit length, positive correlation can be observed on the whole. In reference to kinds of rocks, the trend is remarkable especially in granites.

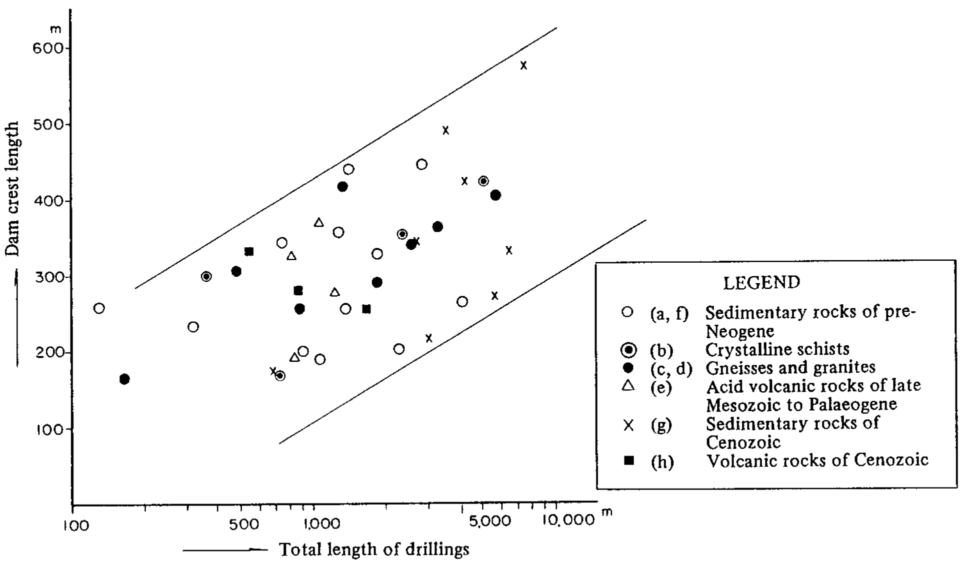


Fig. 6 Relation between dam crest length and the quantity of drilling investigation

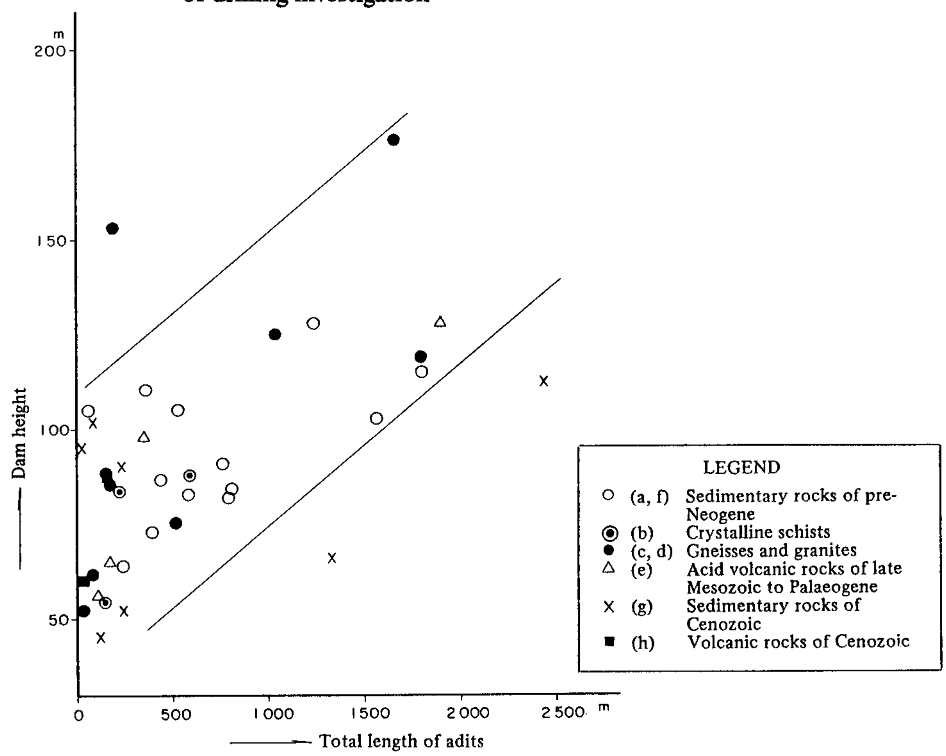


Fig. 7 Relation between dam height and the quantity of adits

(3) Investigations and tests on rock properties

Fig. 8 shows the numbers of the dam cases where particular investigations or tests were conducted on certain rock properties. As for investigations, drillings and adits were comparatively often applied to investigate the rocks with many joints and the rocks with distinct faults, and drillings and seismic prospectings were well used to investigate the extremely weatherd or altered rocks and unconsolidated deposits. Furthermore, for the unconsolidated deposits, test pits were also well used.

As for the field tests, common test items which are mainly applied for the study of mechanical stability are only listed herewith.

Surveys and tests \ Rock properties / Nos. of dams		① Rocks with many joints, rocks with open cracks	② Rocks with distinct faults	③ Extremely weathered or altered rocks	④ Soft rocks	⑤ Unconsolidated deposits
		23	26	16	5	24
Investigation	Seismic prospecting	1	2	4		5
	Drilling	3	7	5		5
	Adit	2	7	2		3
	Test pit		2	1		5
	Trench			1		
Tests	In-situ plate bearing test	1	8	8	3	5
	Drill hole bearing test		3	3	2	1
	In-situ shear test		2	3	2	4
	Triaxial compression test and direct shear test in laboratory		4	3	1	4
	Others	· Swedish sounding	· Swedish sounding · Pull-out test of steel bar · Standard penetration test	· Standard penetration test · Percussion type bearing test	· Piping test · Swelling test by absorption	· Pumping test · Static friction test · CBR test · Standard penetration test

Fig. 8 Investigations and tests conducted in relation to rock properties (including multiple counting)

In the four categories of rock properties excluding the rocks with many joints, in-situ plate bearing test was made in many cases to examine deformation of the foundation rocks. Shearing test was performed relatively often in soft rocks and unconsolidated deposits. (including gravels)

With regard to laboratory tests, triaxial compression test and direct shear test were conducted in many cases for the undisturbed samples obtained from faults, weathered or altered zones and unconsolidated deposits. These cases include 2 cases of large-scale direct shear test for river bed gravels.

As regards "others" test item, standard penetration test was conducted relatively in many cases to know the approximate bearing strength. Special tests conducted include pull-out tests to examine the shear strength of faults, swelling tests for soft rocks, pumping tests to examine the permeability of river bed sediments or ancient valley fills, and so on.

5. Outline of treatment

Main treatments taken for the respective rock properties are shown in Fig. 9. Before these treatments were taken, careful surveys and tests had been carried out to know the distribution and properties of the foundation rocks. The mechanical stability and seepage problems were studied based on those results. More than one method of treatments shown in Fig. 9 were often applied in a dam case, being overlapped.

Treatments \ Rock properties (Nos. of dams)	① Rocks with many joints, rocks with open cracks	② Rocks with distinct faults	③ Extremely weathered or altered rocks	④ Soft rocks	⑤ Unconsolidated deposits: Core zone *	⑤ Unconsolidated deposits: Rock zone
	23	26	16	5	9	15
Curtain grouting & blanket grouting	19	18	11	3	3	
Slush grouting	5	2	1		1	
Re-placement: With core material or filter material		4				
Re-placement: With concrete	1	10				
Underground concrete wall					3	
Excavation and removal (whole or a part)	2	3	11	1	6	
No treatment (left intact)						15
Others	· Fillet-type core embankment · Drain pipe	· Control works for the slope stability at the back of a spillway	· Drain tunnel for the stability of the cut slopes *	· Concrete spraying · Drain work		

(Notice) *; Including deep portion of the foundation

Fig. 9 Rock properties and applied treatments (including multiple counting)

1) Rocks with many joints, rocks with open cracks:

Grouting work to form an impervious zone was mainly adopted (chemical grouting was used together in 3 cases). For this purpose, grout tunnels of two stages were opened in 1 case. In another case, impervious core was embanked like a fillet to widen the basement of the core zone, and in another case, drain pipes were installed. The rocks with open cracks caused by creeping on a slope were removed by excavation in 2 cases.

2) Rocks with faults:

The treatment methods were studied, considering not only seepage problem but also mechanical stability. Grouting (chemical grouting was used together in 1 case) and replacement with core material or concrete were chiefly applied. In grouting, vertical shafts were opened together with tunnels in 1 case, and type of inspection gallery at fault zone was shifted from culvert type to tunnel type in another case.

Removal by excavation was performed in the sites consisted of schistose rocks. Fault zones dipping toward the valley on a slope were completely removed with overlying rocks, and fractured zones dipping toward the hillside were removed together with loose rocks around them by deep excavation in some cases.

In order to improve the slope stability after impounding, where faults were developed in parallel to the inclination of the slope at the back of the spillway, mass concrete walls, PS anchorings, drain tunnels and drain boreholes were executed together in one case.

3) Extremely weathered or altered rocks:

In view of mechanical stability, removal by excavation was mainly adopted in many cases.

Decomposed granites were removed in the foundation even for the rock zone in 1 case. In case those rocks were used as the foundation for the core zone after examining the bearing capacity, grouting was applied in order to improve the watertightness. However, as the rocks with these properties were generally soft, prudent considerations were paid on the proper grouting pressure. In order to highten the grouting effects, chemical grouting was used together with cement grouting in 2 cases, and grout tunnels were provided in 4 cases.

In 1 case, drain tunnels of two stages were constructed in order to secure the slope stability of the spillway during excavation by drawing down the groundwater table.

4) Soft rocks:

Careful considerations were usually paid on the grouting work against seepage. Namely, grout tunnels of two stages were provided for careful execution of grouting, the grouting pressure was severely controlled considering the critical pressure of the foundation, or horizontal drain holes were drilled from an inspection gallery in addition to application of colloid cement for grouting. Partially distributed soft tuffs were entirely removed from a facing type cutoff foundation in one case.

In another case, mortar was sprayed on the excavated surface of soft mudstones on the foundation for the rock zone, in order to prevent the foundation from slaking.

5) Unconsolidated deposits:

In most cases, unconsolidated deposits on the foundation of the rock zone were used directly as the foundation after confirming the bearing capacity, but those of the core zone were mostly removed by excavation. However, when they were left on the foundation of core zone, any other technique for water intercepting than grouting was often used (i.e. underground concrete core wall . . . 3 cases, clay blanket . . . 1 case). For the unconsolidated layer underlying deeply inside the foundation (i.e. old river bed gravels or old talus deposits on the unconformity), careful grouting was carried out from grout tunnels in 2 cases.

Part 2 Design and Practice of Blanket Grouting

1. General

Grouting operations applied on fill dam foundation are devided into blanket grouting and curtain grouting. Blanket grouting is performed primarily for the purpose of improving the impermeability of the upper part of the foundation under an impervious core. And, if necessary, grouting around the inspection gallery in the core zone and consolidation grouting to treat fault zone are also applied, under some special conditions found in the foundation. These gallery grouting and consolidation grouting are also included in the terms of "blanket groutings" for convinience's sake in this report.

2. Planning and designing of the blanket grouting

(1) Area to be grouted

The blanket grouting is usually used in combination with the curtain grouting which is provided to form an impermeable zone in the depths. Ordinarily, one to a few rows of the blanket grouting are provided at upstream and downstream sides of the curtain grouting.

As shown in Table 1, however, the blanket grouting has not always been carried out at the old fill dams. At dams built before 1965, the foundations were treated only with the curtain grouting, and the blanket grouting was apparently provided only partly at places where water flowed out and the area where the geological condition was locally defective.

Table 1 Change of the arrangement of grout holes

Arrangement of grout holes \ Year			before 1965	1966 ~ 1970	1971 ~ 1975	1976 ~ 1980	after 1981
Practice from foundation ground	No blanket grouting		4				
	At places where water flowed out and parts where geological condition was locally defective		3				
	Vertical type	one row of holes each upstream and downstream of the curtain		3	4	1	1
	Radial type				1	1	
	Vertical type	the stagger type of the multiple arrangement			1	9	3
	Vertical type	the lattice type of the multiple arrangement				2	
Practice from inspection gallery	Vertical type			1	1		
	Radial type			1	4	8	2
Combined type	Vertical type combined with radial type					6	2

Notes; Number of dams is 37. In case of being combined vertical type with radial type, it is counted in vertical, radial, and combined type.

Afterwards, since about 1966, the number of cases increased in which a row of comparatively shallow grouting in the vertical direction is made each upstream and downstream of the curtain for the purpose of improving impermeability of the upper part of the foundation. When the inspection gallery was provided, grouting was made radially from there.

On the other hand, since about 1976 the number of rows of provided blanket grouting has in-

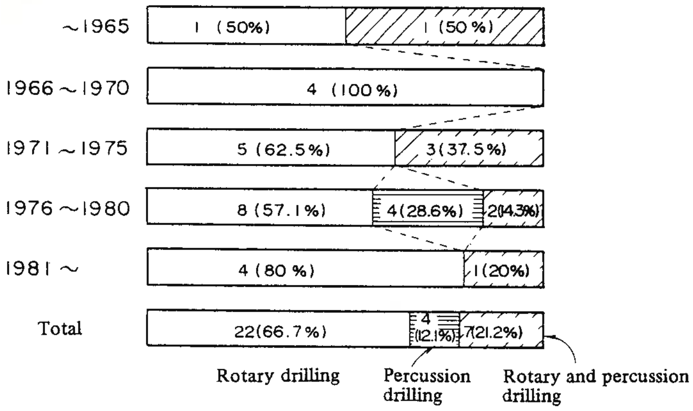


Fig. 7 Method of drilling

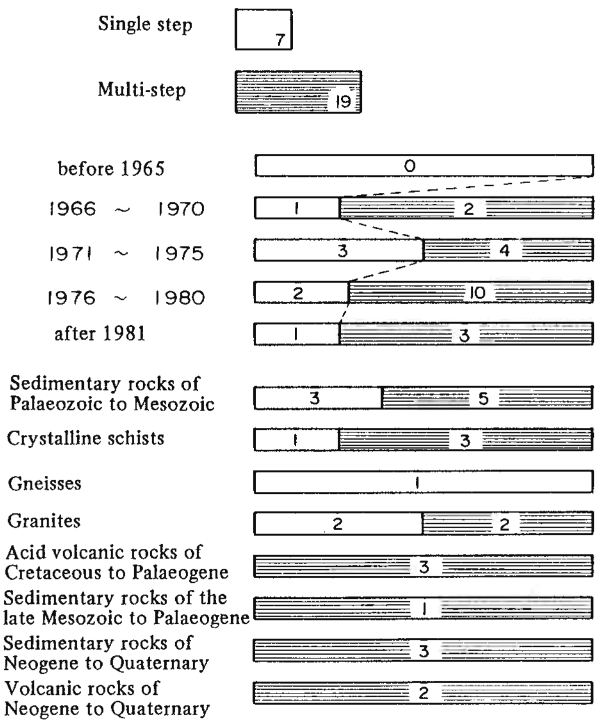


Fig. 8 Pressure stage of water test

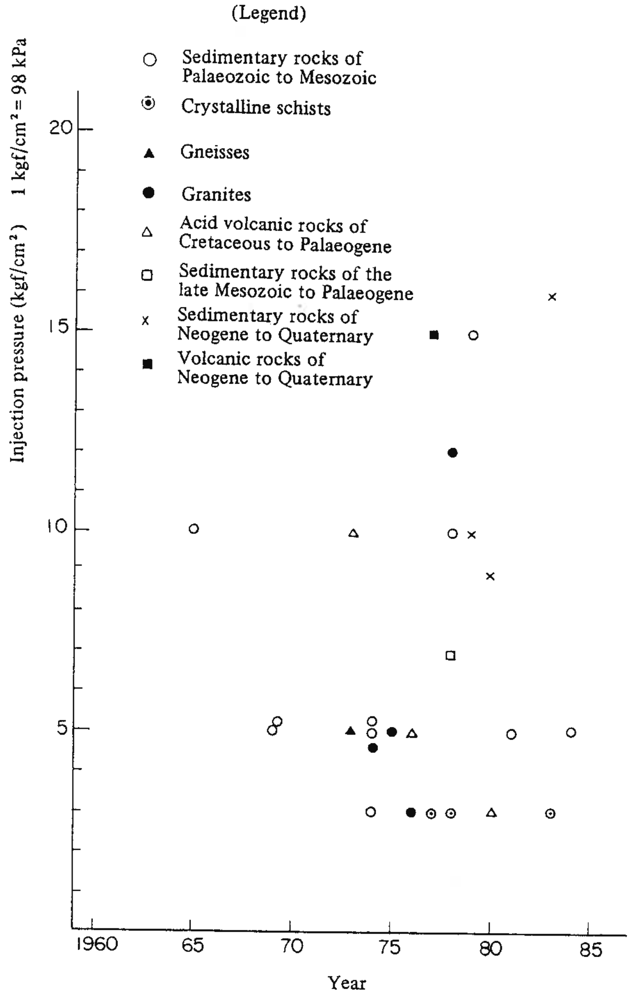


Fig. 9 Injection pressure in the water test

20 dams the test was performed for all the holes. In view of the purpose of water test, it is better to perform the test for all the holes.

b) Steps of injection pressure

As is shown in Fig. 8, single-step procedure and multi-step procedure are applied, but the latter is used for the most cases.

c) Injection pressure

The maximum injection pressure is in the range from 3 kgf/cm^2 to 16 kgf/cm^2 (294 KPa to 1568 KPa), in which in particular the pressure of 3, 5 and 10 kgf/cm^2 (294, 490 and 980 KPa) is frequently used. (Refer to Fig. 9)

d) Injection time

Most of the injection time for one step of injection pressure is 10 minutes.

creased in some dams, one has begun to see such cases as making blanket grouting over the whole face of the core foundation and the combination of blanket grouting arranged vertically in the upstream and downstream of the curtain and the one arranged radially from the inspection gallery. This resulted in the appearance of dams in which the total length and density of blanket grouting (total length of grout holes/maximum width of core x crest length) have drastically increased. (Refer to Figs. 1 and 2.)

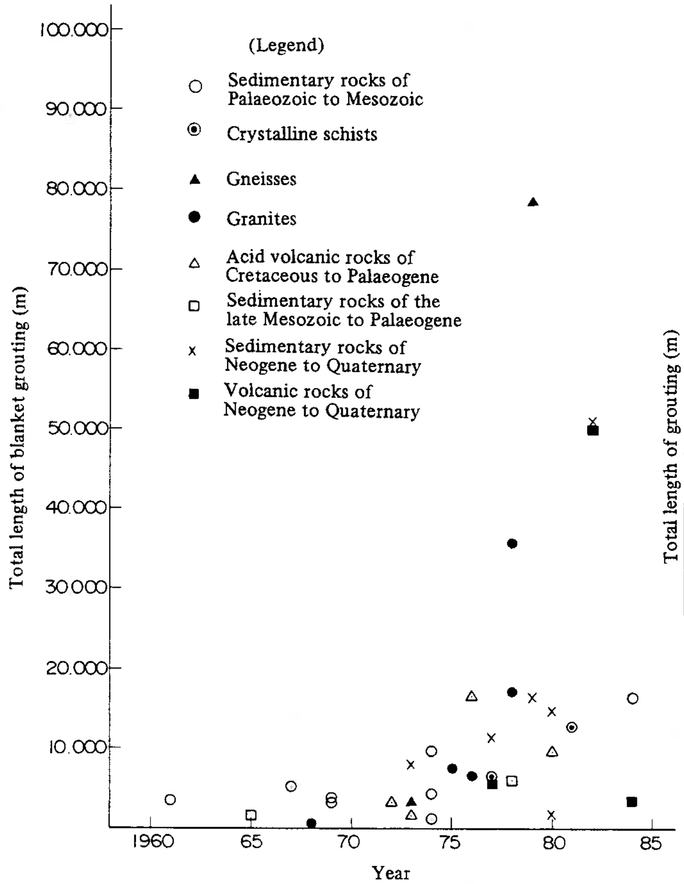


Fig. 1 Chronological change of the total length of grouting

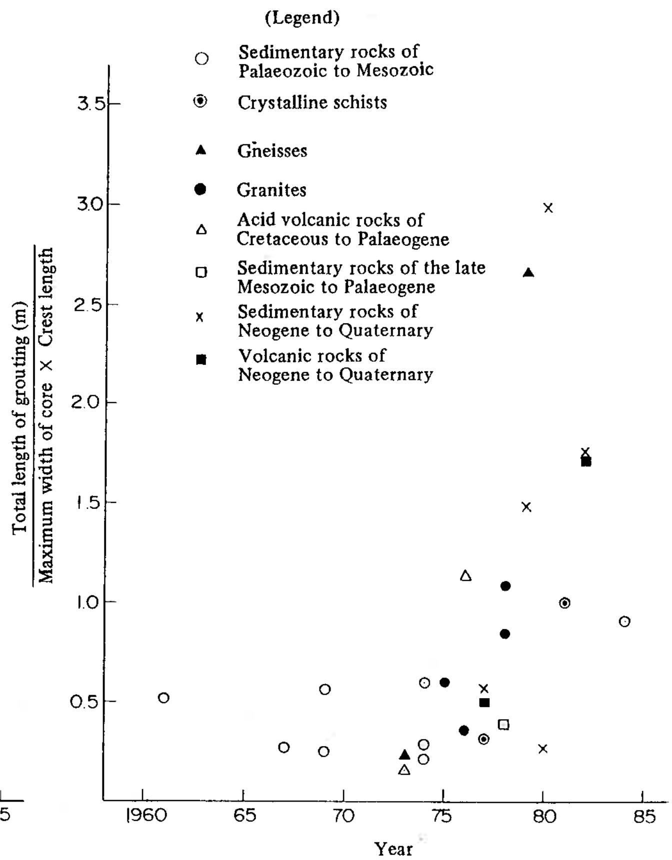


Fig. 2 Chronological change of the density of grouting

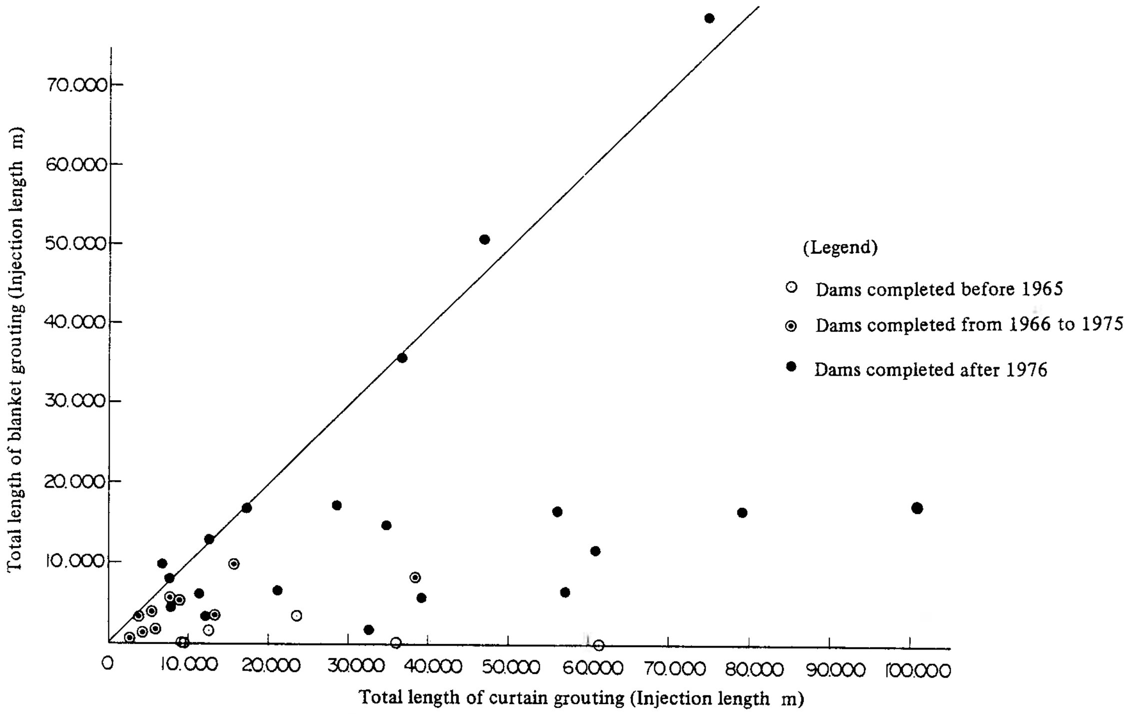


Fig. 3 Comparison between the total length of curtain grouting and of blanket grouting

(3) Grout equipments

a) Grouting plant system

The number of the cases of central plant system is 18, that of the cases of decentralized plants is 13, and that of the cases of combining these is one. In selecting the system of grouting plant, it seems that the optimum system is selected for each project depending on the practice conditions at the each dam site.

b) Pipe arrangement and grout pump

As is shown in Fig. 10, the combination of the single pipe arrangement and discharge adjusting type grout pumps and the combination of circulating pipe arrangement and fixed discharge type grout pumps account for the most part.

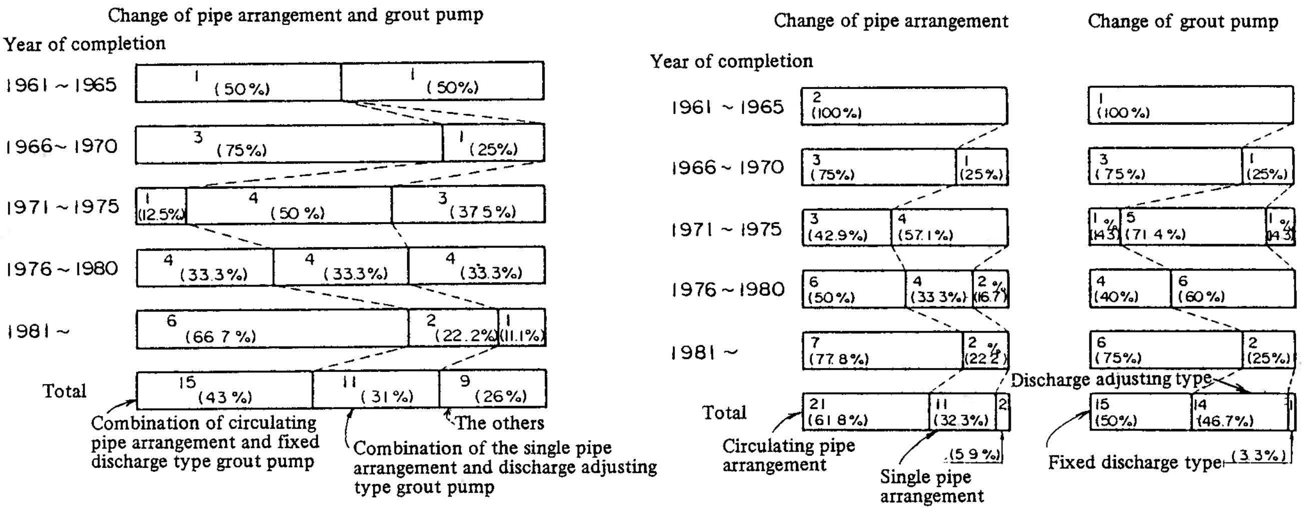


Fig. 10 Change of pipe arrangement and grout pump

(4) Grouting specifications

a) Time of injection

Usually blanket grouting is performed prior to the dam body embankment. In the case of grouting from the inspection gallery, grouting is made in parallel with the embankment or after the completion of embankment.

b) Injection method

Among the 32 dams surveyed, there have been 8 cases of single-step grouting (7 cases of single-step grouting, 1 case of single-step and stage grouting), 27 cases of multi-step grouting. Among the multi-step grouting, there have been 6 cases of packer grouting (4 cases of packer grouting, 2 cases of packer and stage grouting), 21 cases of stage grouting (18 cases of stage grouting, 2 cases of stage and packer grouting, 1 case of stage and single-step grouting).

c) Injection pressure

The inejction pressure ranging from 1 to 5 kgf/cm^2 (98 KPa to 490 KPa) is frequently used in the first stage as shown in Table 2, but has no definite relation with the method of surface treatment and geological conditions.

The maximum injection pressure is plotted against hole depth in Fig. 11. There can be found some tendencies: All the grout pressures are bigger than P = 0.45 h: The pressures in gneisses and granites are P = h: The pressures in crystalline schists and sedimentary rocks of Neogene to Quaternary are between P = 0.45 h and P = h. But injection pressure is usually specified by the results of the preliminary test grouting and adjusted accordingly, if necessary, in the course of the grouting work.

Table 2 Injection pressure at the first stage

Method of surface treatment		Injection pressure in the first stage (kgf/cm²) 1 kgf/cm² = 98 KPa								
		0	1	2	3	4	5	5 ~ 10	10 ~ 15	15 ~ 20
Practice from foundation ground	No treatment		○		○	○	○ △ ×			
	Slash grouting		◉	◉	●	●		●		
	Cover rock		× △	×	× × △		×			
	Cap concrete		▲ × ◉	△ ▲ △ ● □ × ×	△ ▲ △ × □ × ○	▲ △	▲ △ ○ ×	▲		
	Gunite-shooting		△	× ×	● ◉ × × ▲ △ × ○	▲	● ▲ ×	■		
Practice from inspection gallery	Core embankment over 10 m				○ ○		○			
	Core embankment over 20 m							▲	▲	▲
	Obscure		×	● ● ×	● ×	● ×	● ◉ × ■	△ □ ×		

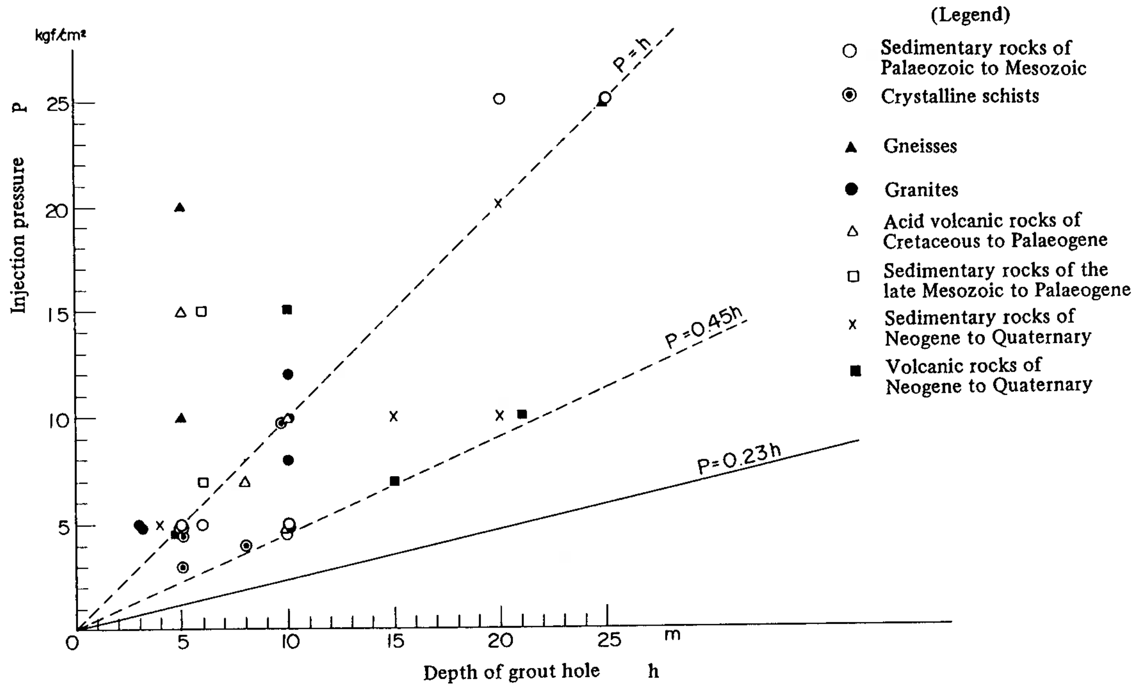


Fig. 11 Relation between the depth of grout hole and the injection pressure

d) Injection speed of grout

As shown in Fig. 12, since about 1975 injection speed of grout has been regulated in addition to the injection pressure. This could be explained by the circumstances that it has become unavoidable since that time to perform grouting into geologically poor base rock to prevent eventual upheaval of the base rock caused by excessive high pressure injection. The maximum injection speed of grout is usually between 2 to 6 ℓ/min/m.

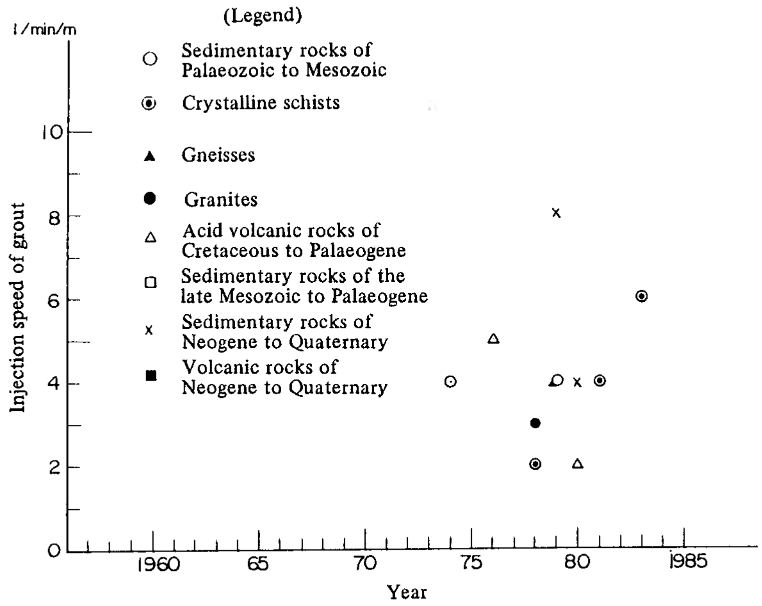


Fig. 12 Chronological change of injection speed of grout

e) Grouting material

Grouting material is mostly normal Portland cement and blast furnace cement, but there are cases in which colloidal cement is used for soft rock. (Refer to Fig. 13)

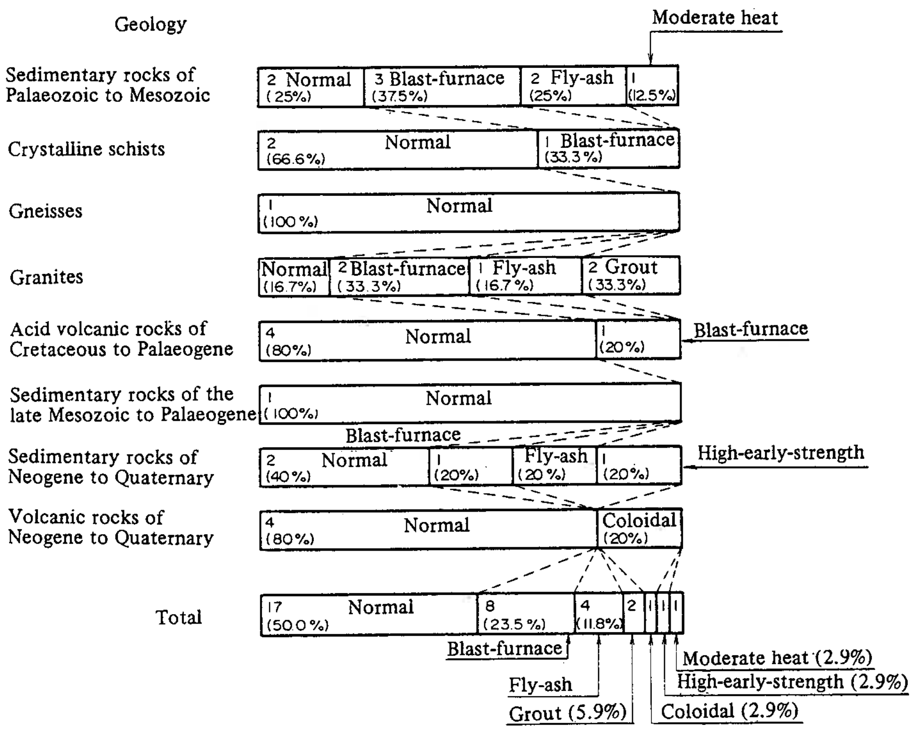


Fig. 13 Grouting material

f) Mix proportion and standard for changing mix proportion

There have been four to nine classes of mix proportion established from thin to dense ones within the W/C ratio of 10 to 1. Most of the starting mix proportion is 10 of W/C ratio, but there are cases with a denser mix proportion.

Table 3 shows four standards how to change mix proportion and number of dams. Among them, those with usual practice are the ones of changing mix proportion according to specified grouting volume and rate of injection, and of combining these two.

Table 3 Method of changing mix proportion

Method of changing mix proportion \ Year		before 1965	1966 ~ 1970	1971 ~ 1975	1976 ~ 1980	after 1981
① Changing mix proportion according to specified injection		1		1	1	1
② Changing mix proportion according to specified grouting volume	Constant volume		1	2	3	
	Specified with each mix proportion				2	2
③ Changing mix proportion according to rate of injection				3	4	1
④ Combination of ①~③			② and③ 3	②and③ 1	② and③ 4	

g) Standard of completion of injection

As shown in Fig. 14, there are many cases in which grouting is completed by "blocking" for 20 to 30 minutes after the injection speed of grout has attained less than 0.2 ℓ/min/m under the specified injection pressure.

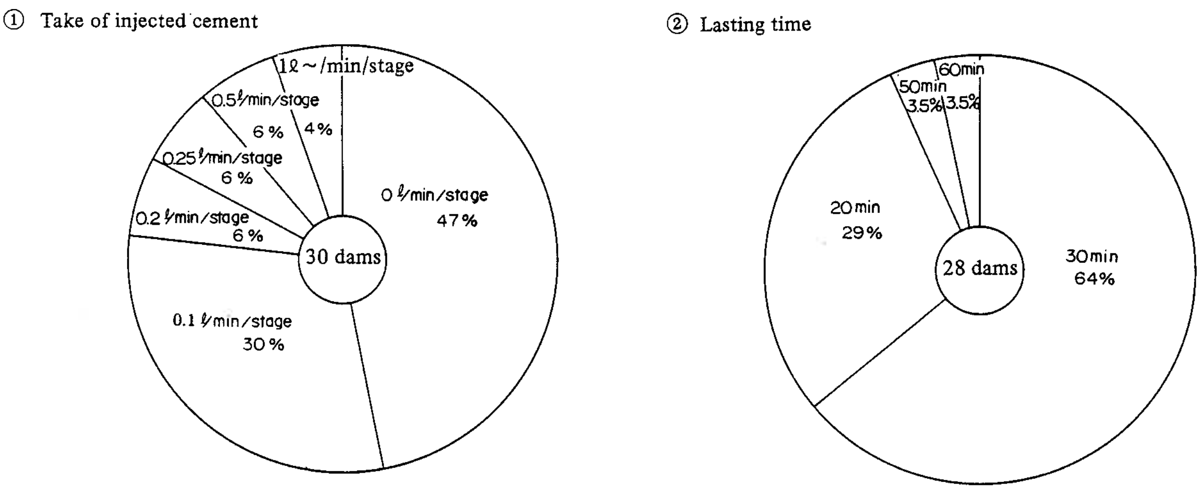


Fig. 14 Standard of completion of injection

4. Results of injection in blanket grouting

(1) Results of injection

Fig. 15 gives the results of cement take per unit length[1] and per unit volume of foundation[2] for each kinds of rocks. As the figure shows, the grout take fairly varies within each of the rock kinds, suggesting that the grout take cannot be presumed by geological classification alone.

(2) Evaluation of the effect

There are three methods of evaluating the degree of improved impermeability: ① one by Lugeon values, ② one depending on the take of injected cement, and ③ the combination of the two. Fig. 16 shows the number of dams, built in every 5-years period since 1965.

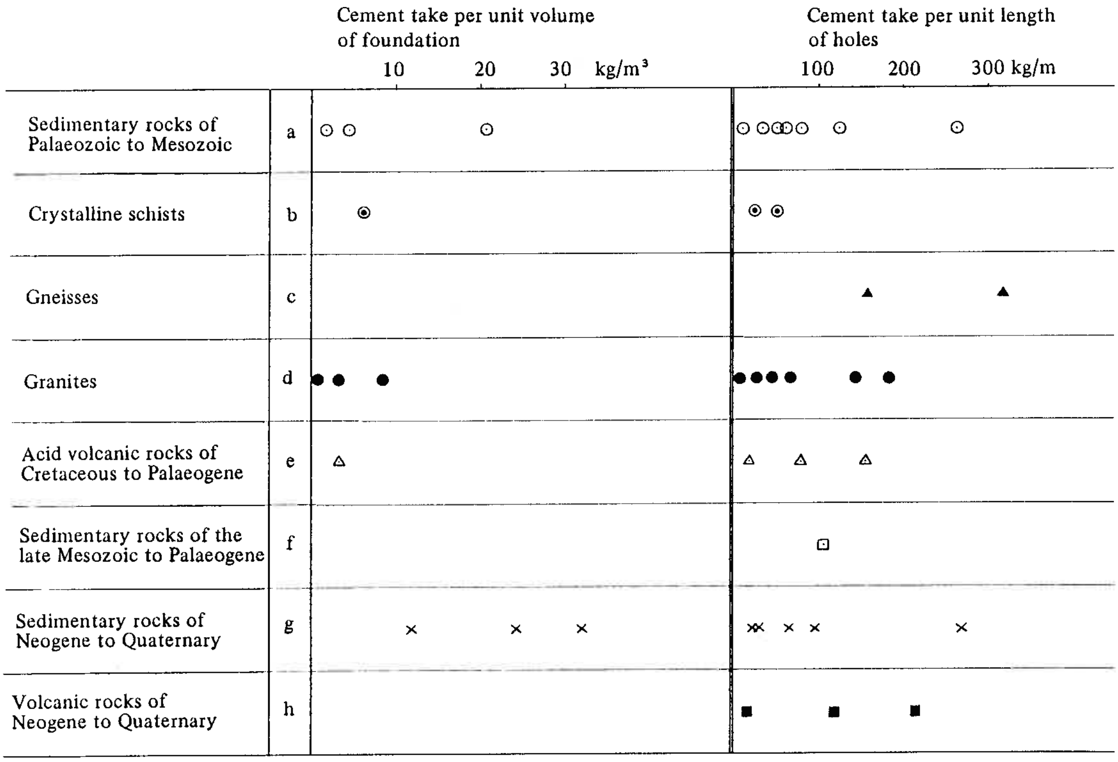


Fig. 15 Cement take for each kinds of rocks

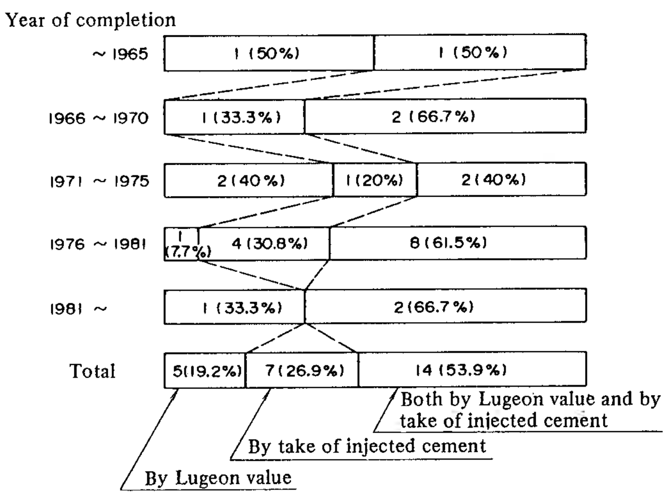


Fig. 16 Method of evaluation of the effect

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1. Grout take per unit length of holes = $\dfrac{\text{Total grout take}}{\text{Total length of holes}}$

2. Grout take per unit volume of foundation = $\dfrac{\text{Total grout take}}{\text{Grouted area} \times \text{Average length of holes}}$

Average length of holes = $\dfrac{\text{Total length of holes}}{\text{Number of holes}}$

Part 3 Design and Practice of Curtain Grouting

1. General

The foundation rock of a dam must have sufficient strength, rigidity and impermeability to secure structural and hydraulic stability of the dam. The force acting on the unit area of the foundation rock of a rockfill dam is smaller than that of a concrete dam, and strength and rigidity of the foundation rock do not come into question except special cases. However, as embankment materials of the rockfill dam have lower resistance against piping by seepage water, impermeability of the foundation rock is highly required. Impermeability of the foundation rock of a rockfill dam is generally improved by blanket grouting and curtain grouting. Curtain grouting is adopted to suppress the seepage flow through the foundation rock. Blanket grouting is adopted to secure the hydraulic stability against piping near the surface of the foundation rock of a rockfill dam.

Planning, design, practice and result of curtain grouting are described in the following articles based on the questionnairing to 44 major rockfill dams.

2. Planning and design of curtain grouting

(1) Planning of curtain grouting

In planning curtain grouting, seepage flow in the foundation rock, which might have influence on the stability of the dam and its foundation rock, must be comprehensively evaluated. It requires surveys for the topographical and geological features of dam site and ground water level and tests for properties of foundation rocks such as permeability and grouting characteristics. Design items must include: (1) area to be grouted, (2) target value of improvement, (3) spacing of grout holes, (4) type of grout and its mixture, (5) sequential procedure of grouting, (6) pressure and rate of grout injection, (7) evaluation method of results. However, seepage flow in the foundation rock is difficult to be estimated theoretically, and design of grouting depends on permeability tests and test grouting at each dam site.

(2) Design of curtain grouting

a) Area to be grouted . . . Fig. 1 shows that the depth of curtain grouting was more decided by the height of the dam than by the properties of the foundation rock such as permeability. Water pressure acting on the foundation rock is determined by the height of the dam and there is a reason why the depth of grouting is decided by the height of the dam. But permeability and other properties of the foundation rock should be also taken into consideration. Recently there are cases where the grouting depth is larger than the height of the dam shown in Fig. 2. It seems that the depth of grouting becomes more decided by the geological properties of the foundation rock. Fig. 3 shows the depth of grouting in relation with geological features. The cases where the depth of grouting is larger than the height of the dam are mainly observed on the sedimentary rocks of Neogene to Quaternary periods.

Fig. 4 shows that in recent years the use of grout tunnel has been prevailing and rim grouting has been also applied to more extended range.

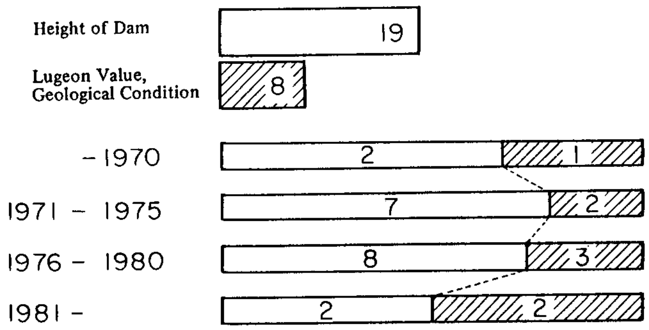


Fig. 1 **Parameter to decide the depth of grouting**

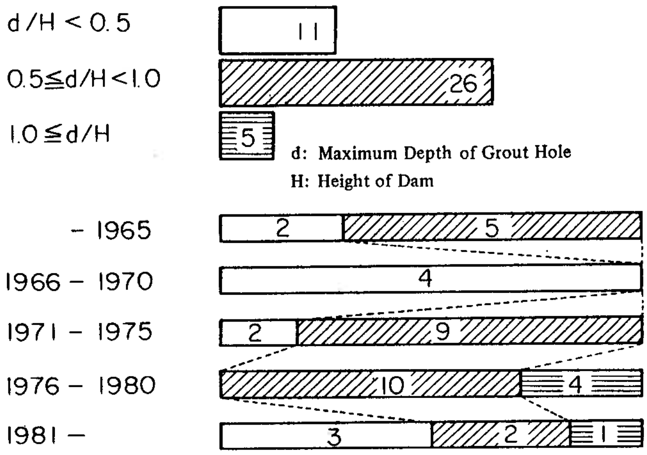


Fig. 2 **Change of the depth of grouting in relation to construction years**

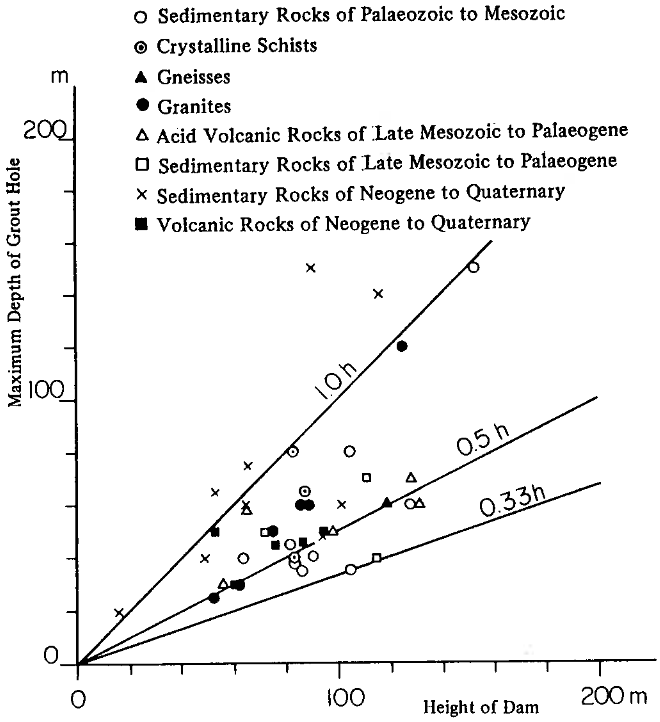


Fig. 3 **Relation between the height of the dam and the depth of grout hole**

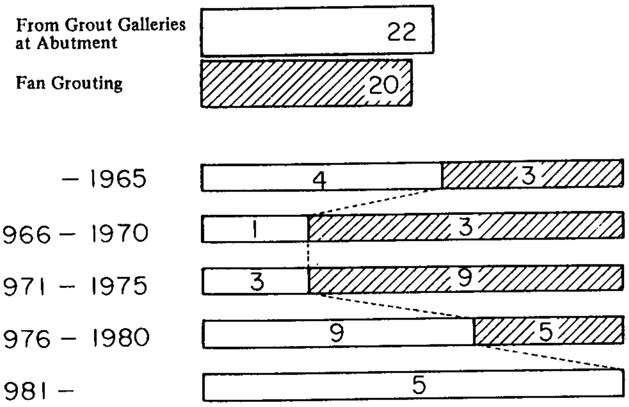


Fig. 4 **Practice of rim grouting**

b) Target value of improvement . . . As shown in Fig. 5, target values of 3 to 5 Lu were widely adopted, and this trend has been almost the same from year to year. The target values of improvement do not show any significant relation with geological features.

c) Rows and direction of grout holes . . . Single row, double row and triple row grouting was adopted in curtain grouting almost at the same rates, but in recent years, plural-row grouting has increased as shown in Fig. 6. Plural-raw grouting was mostly used for sedimentary rocks and volcanic rocks of Neogene to Quaternary. This seems to reflect the difficulty of grouting in both geological conditions.

Most grout holes were drilled vertically as shown in Fig. 7. There were also many case where grout holes were drilled both vertically and obliquely. There was no clear correlation between the direction of grout holes and geological features. The direction of holes seems to have relation rather with such topographical conditions as the slope of the abutment.

d) Injection pressure of grout . . . The relation between injection pressure and the hole depth is shown in Fig. 8. (The depth on the axis of ordinate does not mean the maximum depth of grout holes, but the depth corresponding to maximum injection pressure.) Injection pressure is higher for the sedimentary rocks of Mesozoic, and lower for the sedimentary rocks of Neogene to Quaternary. This seems that critical pressure is considered. Fig. 9 shows the changes of maximum injection pressure in relation to construction year. In 1970s, higher injection pressures were often adopted, but recently the maximum injection pressure has been lowered. This shows that not only grouting efficiency by raising injection pressure but also critical pressure were taken into account in practice.

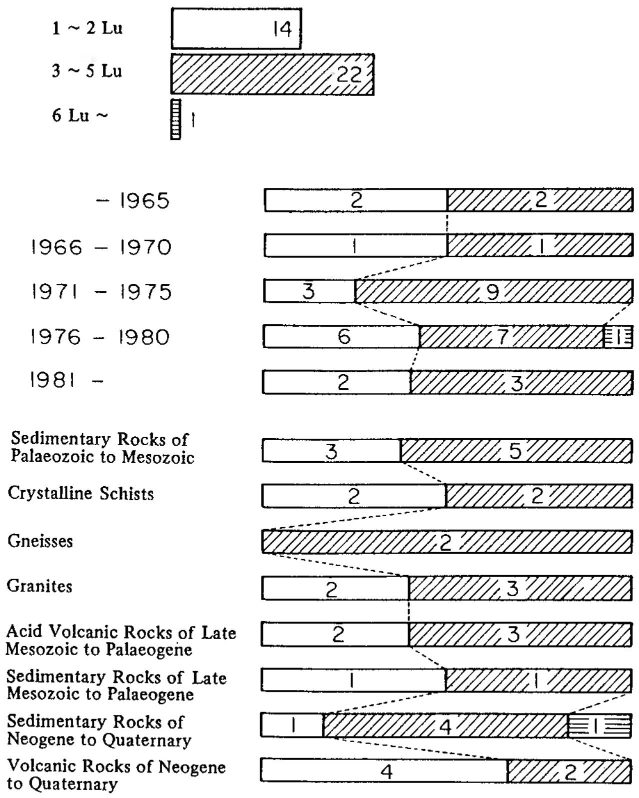


Fig. 5 Target values of improvement

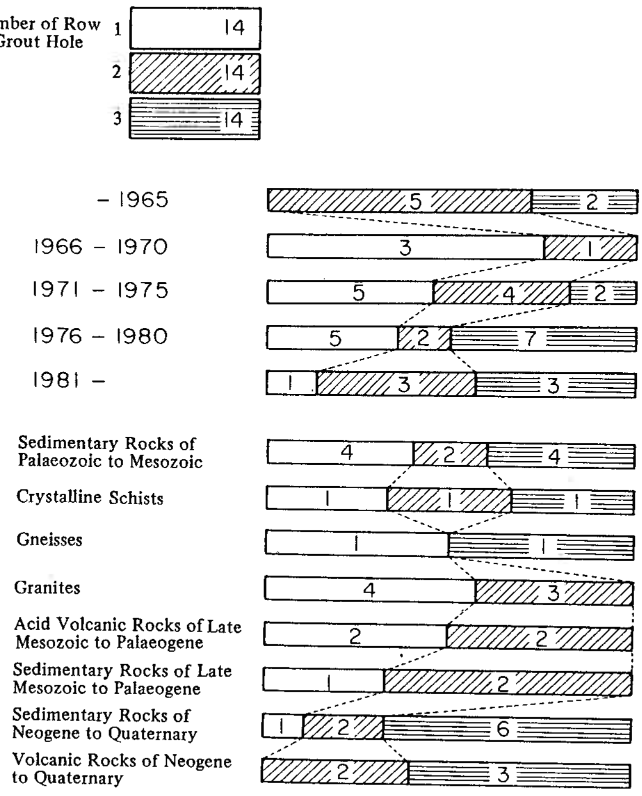


Fig. 6 Rows of grout holes

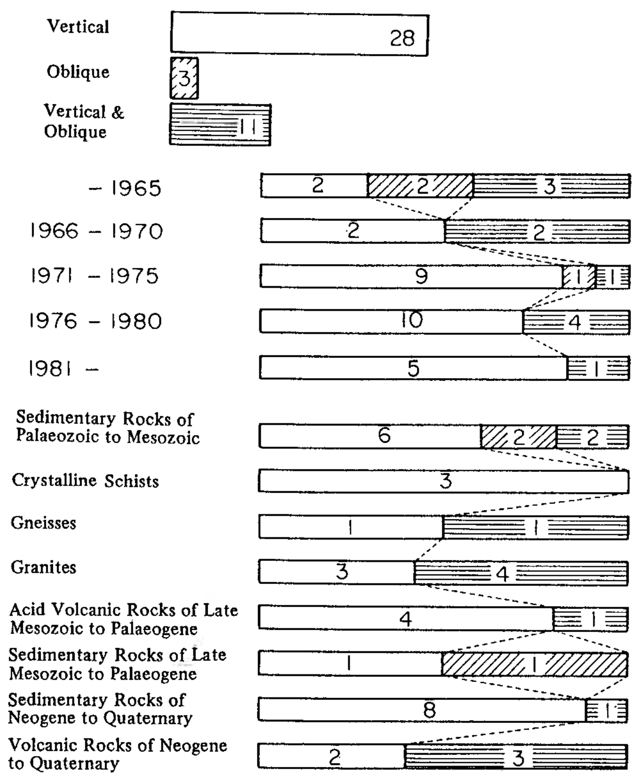


Fig. 7 Directions of grout hole drilling

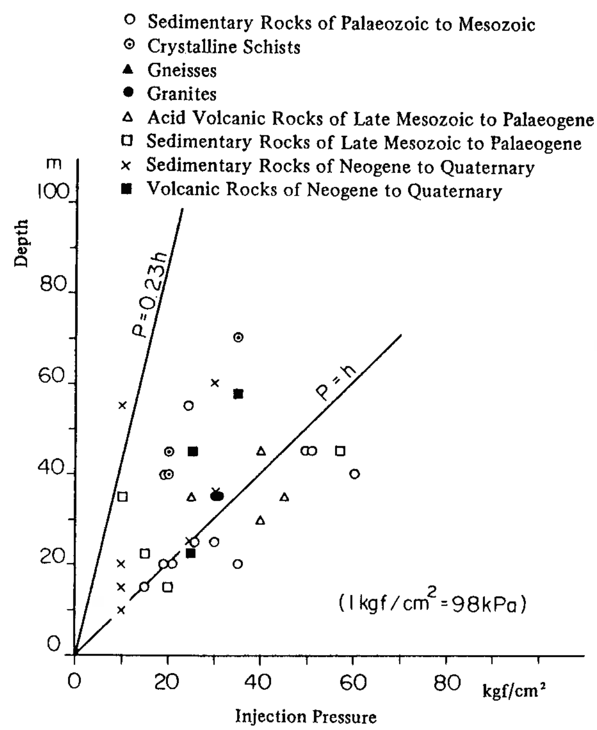


Fig. 8 Relation between injection pressure and the depth of grout hole

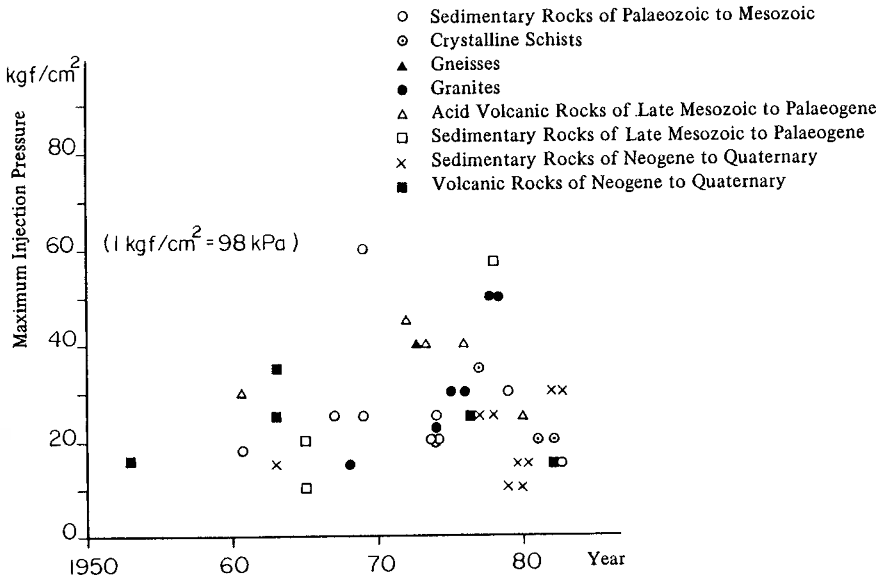


Fig. 9 Change of maximum injection pressure in relation to construction years

3. Practice of curtain grouting

(1) Practice of curtain grouting

The design theory of grouting cannot be said to have been sufficiently established yet, and in general, the specifications of grouting are modified to suit the conditions of the site according to the results of grouting.

(2) Grouting equipments

a) Drilling machine . . . Rotary drilling machines are generally used for curtain grouting. There were four cases where percussion drilling machines were used together.

b) Grouting plant . . . A central grouting plant was equipped at 23 dams, decentralized grouting plants at 14 dams, and both combined at 5 dams. There was no clear difference in the selection of the type from year to year. The respective types seem to have been decided by the practical conditions of each dam site.

c) Grout pump . . . Fig. 10 shows that the discharg adjusting type used in 1970s was mostly substituted by the fixed discharge type in recent years. It is surmised that the electromagnetic flowmeter is

(3) Water test

a) Test holes . . . At 32 dams out of 41, all holes were water tested. At 9 dams which was constructed in early days, only pilot holes were tested. All grout holes should be tested to know permeability of the foundation rock.

b) Steps of injection pressure . . . Fig. 11 shows single injection pressure, ascendant multiple injection pressures and both accendant and descendant multiple injection pressures are adopted in water tests. Recently, multiple injection pressures were often adopted because the measurement of critical pressure is important. There was no clear relation between the steps of injection pressure and geological features.

c) Injection pressure . . . Fig. 12 shows that injection pressure of the water test was changed according to the depth of a hole at many dams, and 10 kgf/cm^2 (980 kPa) or less than 10 kgf/cm^2 (980 kPa) are usually adopted. Maximum injection pressure seems to be decided at the level less than critical pressure.

d) Injection time . . . The injection time was usually 10 minutes at each testing pressure, and especially in recent years time more than 10 minutes is not adopted except a few case. However, the reason for deciding the injection time was not quite clear.

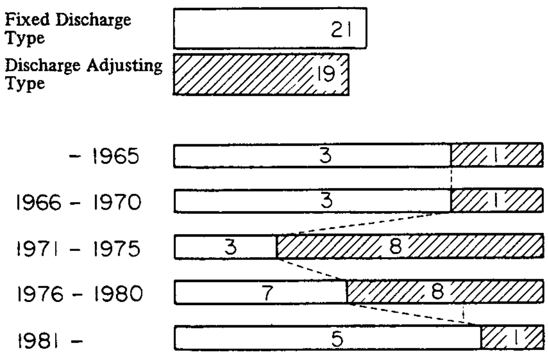


Fig. 10 Types of grout pumps

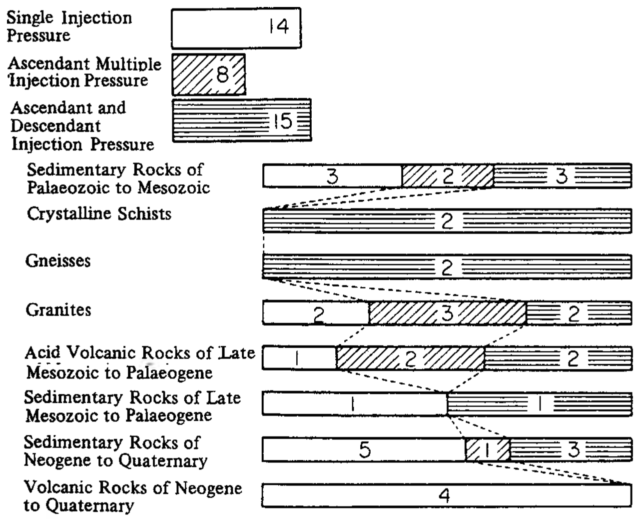


Fig. 11 Steps of injection pressure in water tests

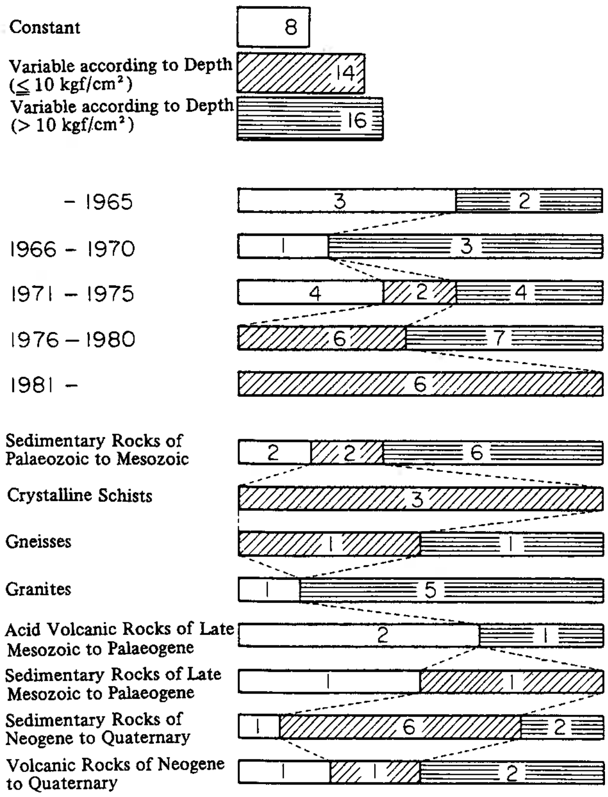


Fig. 12 Injection pressure in water tests

(4) Grouting specifications

a) Stage of practice . . . Recently inspection gallery was provided in many dams and curtain grouting was practiced after the embankment had been filled up to some extent because it make possible to adopt higher pressure without causing upheaval of the foundation rock.

b) Material . . . As shown in Fig. 13, colloidal cement was sometimes used for the foundation rock difficult to be improved by grouting. The use of blast furnance cement seems to depend on local conditions.

c) Procedure of grouting . . . As shown in Fig. 14, stage grouting was usually adopted. Packer grouting was adopted at dams on hard rocks. At some dams, both grouting were adopted at the late stage of practice or to the foundation rock with good geological conditions.

d) Change of mix proportion . . . The first mix proportion of grout was W/C = 10 in many cases. There were many patterns before 1975 such as W/C = 20 and W/C = 4. In recent years mix proportion started from W/C = 10 to 8 at many dams. The first mix proportion was selected by Lugeon values at some dams.

e) Control of injection speed of grout . . . As shown in Fig. 15, injection speed of grout was controlled at recent dams. It is surmised to be controlled to prevent upheaval of the foundation rock. The relation between injection speed of grout and geological features does not seem clear.

f) Final spacing of grout holes . . . As shown in Fig. 16, in recent years the final spacing of grout holes became narrow, which suggests bad geological conditions.

g) Completion of injection . . . Injection was generally completed when injection speed of grout was 0 to 0.2 ℓ/min/m, and pressure was sustained for 20 to 30 minutes.

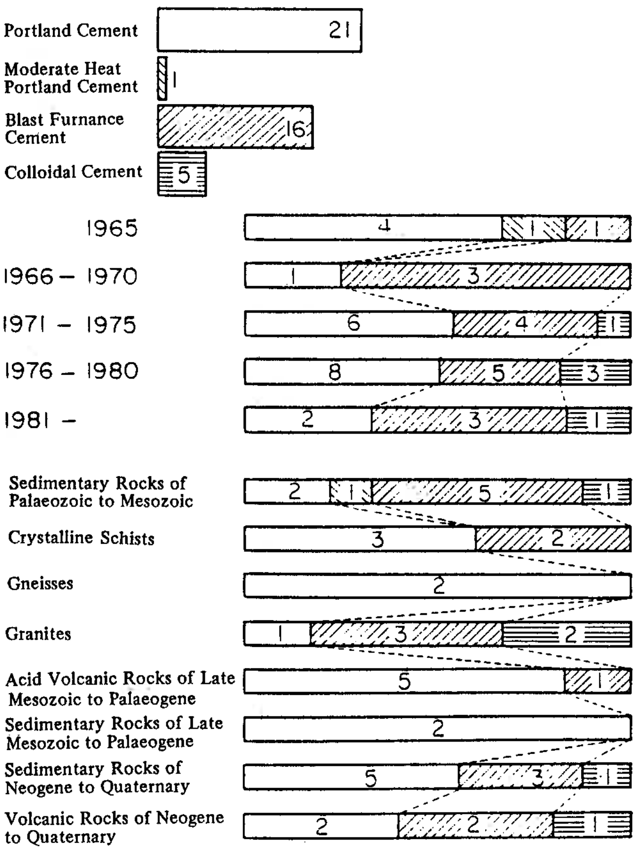


Fig. 13 Type of grout

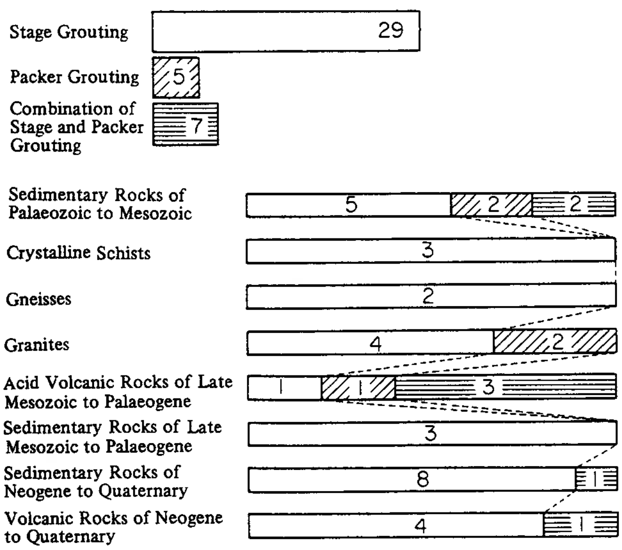


Fig. 14 Injection methods

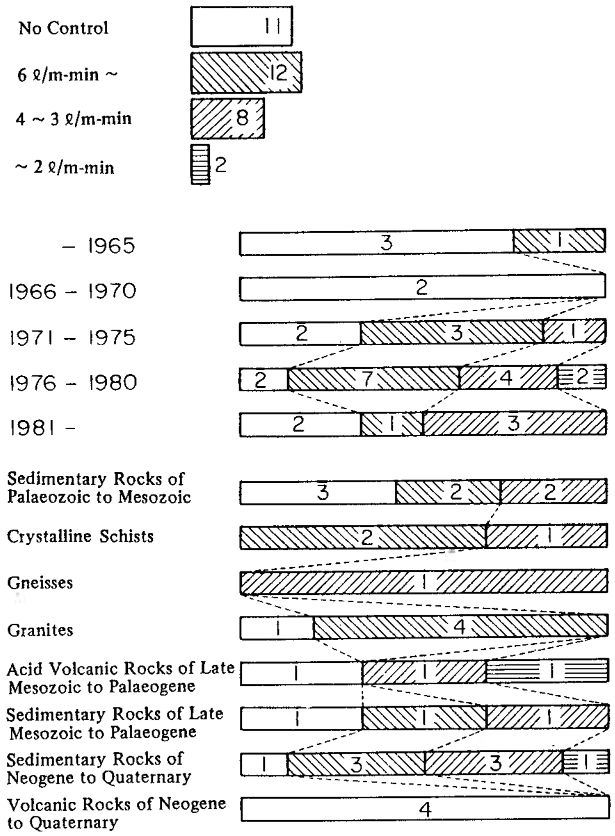


Fig. 15 Control of injection speed of grout

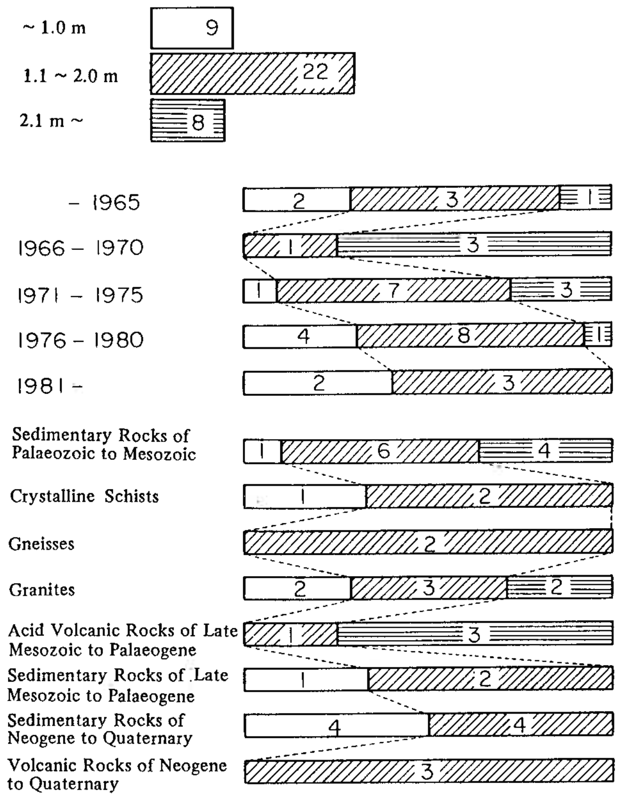


Fig. 16 Final spacing of grout holes

4. Evaluation of the results of curtain grouting

(1) Evaluation of the results of curtain grouting

It is difficult to accurately predict improvement of impermeability of the foundation rock by grouting at the design stage. The result of grouting should be always checked to confirm the improvement of impermeability. Additional grouting should be placed when impermeability is not improved.

(2) Evaluation method of the results

a) Evaluation of the results . . . Improvement of impermeability of the foundation rock by curtain grouting is evaluated related to the change of grout take of each hole by split spacing methods and the Lugeon values of check holes. As shown in Fig. 17, impermeability was evaluated related to Lugeon values or combination of Lugeon values and the amount of grout take at many dams. In recent years, the latter was generally adopted.

(3) Results of grouting

a) Total hole length . . . The change of the total hole length of curtain grouting in relation to the construction years is shown in Fig. 18. The total hole length is affected by the height of the dam, crest length, etc. to some extent. However in recent years, the total hole length increases. The total hole length increases in crystalline schists, sedimentary rocks of Neogene to Quaternary and volcanic rocks.

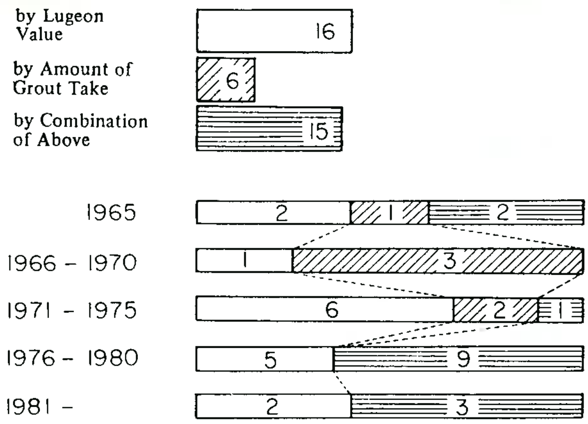


Fig. 17 Evaluation methods of results

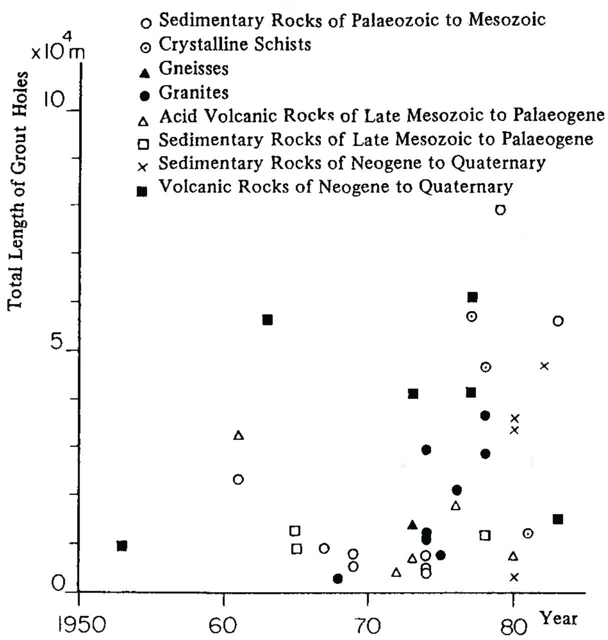


Fig. 18 Change of total hole of curtain grouting in relation to construction years

b) Cement take per unit length of grout hole . . . Fig. 19 shows the cement take per unit length of grout hole for the types of rocks. Cement take was larger in gneisses and the volcanic rocks of Neogene to Quaternary. On the contrary, it was smaller in crystalline schists and the sedimentary rocks of Neogene and Quaternary. Fig. 20 shows the relation between the total hole length and total cement take. In the volcanic rocks of Neogene to Quaternary, both the total hole length and the total cement take show large values, suggesting high permeability. In the sedimentary rocks of Neogene to Quaternary, the total hole length was long compared with total cement take and it is surmised to suggest that the improvement by grouting is difficult.

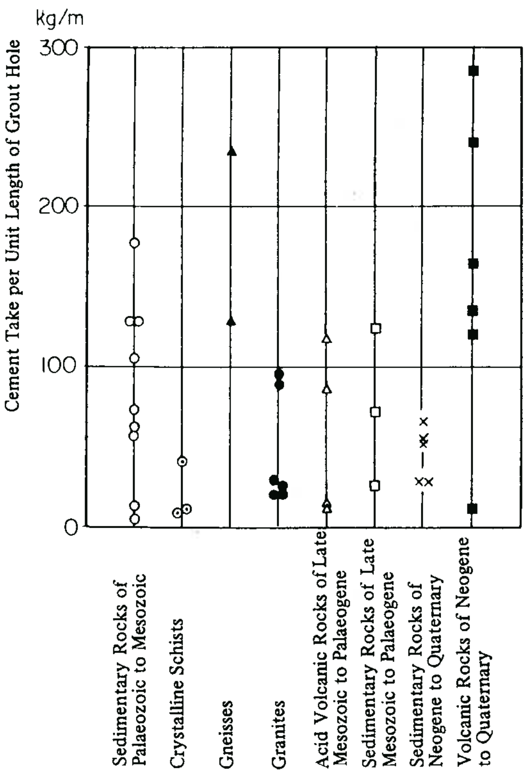


Fig. 19 Cement take per unit length of grout hole for types of rock

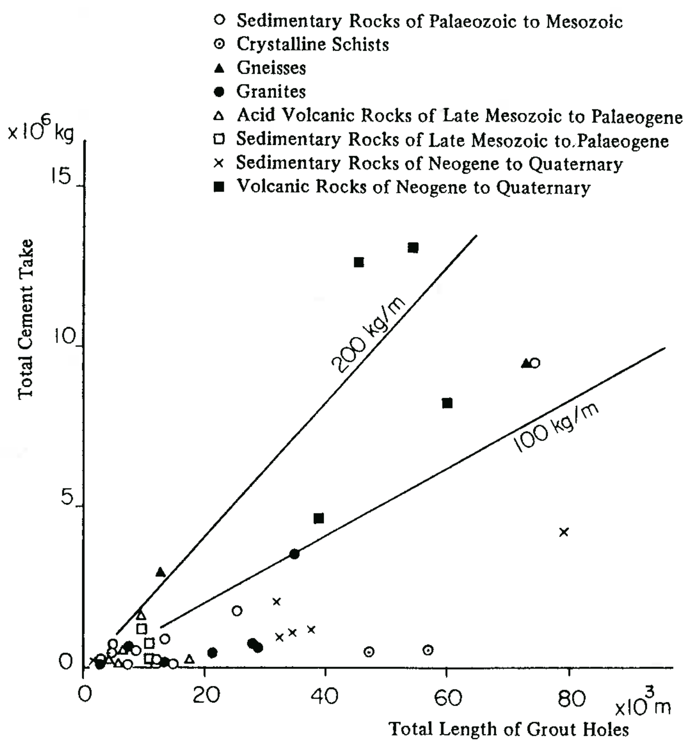


Fig. 20 Relation between total hole length and total cement take

c) Cement take per unit area of grout curtain . . . Cement take per unit length of grout hole is affected by spacing of grout holes and it does not show the real characteristics of the foundation rock for grouting. Thus the grouting characteristics of the foundation rock are evaluated by cement take per unit area of grout curtain. As shown in Fig. 21, cement take per unit area of grout curtain was larger in the volcanic rocks of Neogene to Quaternary and gneisses. On the other hand, the sedimentary rocks of Palaezoic to Mesozoic which have large cement take per unit length of grout hole in Fig. 19, have relatively small cement take per unit area of grout curtain in Fig. 21. It is surmised that high injection pressure used in the sedimentary rocks of Palaezoic to Mesozoic shown in Fig. 8, extends the reach of grout.

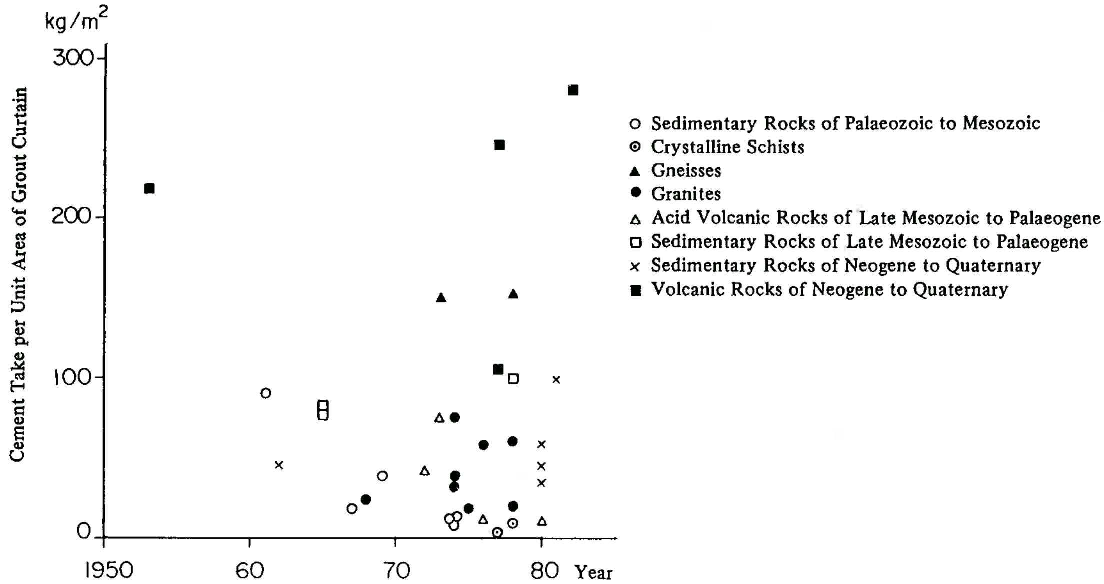


Fig. 21 Cement take per unit area of grout curtain

5. Special grouting and special foundation treatment

(1) Methods of special grouting and special foundation treatment

The methods of special grouting and special foundation treatment are classified as follows (see Table 1).

a) Other methods than grouting . . . Underground concrete wall, etc.

b) Grouting by any other material than cement . . . Chemicals, bentonite, mortar, etc.

c) Special grouting methods . . . Sleeve grouting (such as Soletanche), slush grouting

d) Others . . . Narrow spacing

(2) Geological characteristics in relation to special grouting and special foundation treatment

a) Rocks with many cracks and joints and rocks with open cracks . . . Cement and chemical grouting was adopted in the sedimentary rocks of the end of Mesozoic to Palaeogene.

b) Faults & fracture zones . . . These are observed in all geological conditions except the Quaternary stratum. In general, cement grouting was adopted, but there were dams adopting chemical grouting together. When it was difficult to hold bore walls, sleeve grouting was adopted.

c) Extremely weathered rocks and altered rocks . . . There were cases with weathered granites. Chemical grouting was adopted.

d) Soft rocks . . . In soft rocks, usual grouting was adopted.

e) Unconsolidated sedimentary stratum . . . In a gravel stratum of Quaternary, an underground concrete wall was constructed.

Table 1 Special grouting and special foundation treatment

Geological Feature			Treatment		
Classification	Rock	Geological Condition	Treatment Method	Area of Treatment	Content
Crack, Joint	□	Weathering, Open Crack	G, C	Depth 2-3 m	650 Holes, 4.2 kg/m
Fault	△	Fault 30-50 m	G	Depth 50 m	
Fault	○	Fault 3.5 m	G, C	Depth 8-10 m	120 Holes, 283 kg/m
Fault	△	Fault 3-15 m	G	Depth 10 m	768 Holes, 23.5 kg/m
Fault	□	Fault 5-10 m	G, C	Depth 7-42 m	71 Holes, 95 kg/m, 48 ℓ/m
Fault	●	Fault 4-14 m	G	Depth 10 m	633 Holes, 10.4 kg/m
Fault	▲	Fault 25 m	G	Depth 40 m	81.7 kg/m
Fault	×	Fault 10 m	G *	Depth 60 m	
Weathering, Alternation	●	Crack, Weathering	C	Depth 10 m	31 Holes, 123 ℓ/m
Weathering, Alternation	●	Crack, Weathering	C	Depth 15 m	265 Holes, 190 ℓ/m
Unconsolidated Sediment	■	Old River Bed Gravel	W	Depth 4 m, Width 2.3 m	
Unconsolidated Sediment	■	Old River Bed Gravel	W	Depth 12-20 m, Width 1 m	
Cavern	○	Cavern in Limestone	G, M	Depth 150 m	50 kg/m

Note) Symbol of Rock
- ○ Sedimentary Rocks of Palaeozoic to Mesozoic
- ▲ Gneisses
- ● Granites
- △ Acid Volcanic Rocks of Late Mesozoic to Palaeogene
- □ Sedimentary Rocks of Late Mesozoic to Palaeogene
- × Sedimentary Rocks of Neogene to Quaternary
- ■ Volcanic Rocks of Neogene to Quaternary

Symbol of Treatment
- G Cement Grouting
- C Chemical Grouting (* Sleeve Grouting)
- W Underground Concrete Wall
- M Mortar Grouting

Part 4 Contact Surface Treatments of Foundation Rock and Design and Construction of Inspection Galleries

1. General

Since the failure of Teton Dam in June, 1976, the safety of fill dams against infiltration breakage has attracted serious concern in Japan.

The cause of the failure of Teton Dam can be classified into two factores,

i) Cracking or hydraulic fracturing of impervious core material.

ii) Piping along the contact between impervious core material and foundation rocks.

As shown in above example, surface rock is directly contact with the core material, and because of this, piping is induced when velocity of seepage flow grew higher than critical cohesive force. Furthermore, if the excavated form of the rocks is not suitable, a sufficient compressive stress may not be transmitted to the core, or differential settlement may be caused, or infiltration breakage may occur. Therefore, it is not too much to say that the contact surface treatment of foundation rock is a key to the safety of a fill dam.

Since it relatively often occurs that dam site must be inevitably selected at a place of poor gelogical conditions, in view of the present situation of fill dam construction in Japan, the contact surface of foundation rock must be carefully treated, to prevent the piping which may occur along the contact between impervious core and foundation rock.

2. Methods of contact surface treatment for foundation rock.

(1) Excavation of riverbed and abutment

Table 1 Excavation standards for core foundations

Excavation Standards		Cases	Remarks
Rock grades	Good portions of Grade B to C	1	
	Grade C	1	
	Grade C_M or more	17	Partially including Grade C_L
	Grade C_L or more	3	
Other expressions	Down to hard rock line	4	Judged in reference to bearing capacity and impermeability
	Controlled in reference to C and ϕ by situs CBR test	1	$\tan\phi = 0.7$, $C = 7$ tf/m^2 Compared with single plane shear tests
	Weathered portions are controlled in reference to N values	1	N > 50, (weathered sandy granite)
	Excavated to rocks	1	
	Top soil, talus, intensively weathered portions and very cracked portions were removed.	5	The base rocks of 5 cases include tuff (2), quartz porphyry (1), dacite (1) and Palaeozoic sedimentary rocks (1)
	Excavated to a specific layer	1	Volcanic mud flow deposits are removed, and excavated to tuff breccia

a) Excavation standards

In principle, core zones are placed on rock foundation. Out of 35 dams, foundation of 17 dams were excavated to bedrocks of Grade C_M, showing the largest number.

Also as for other standards, design consideration was given to enhance the impermeability by grouting and to obtain required bearing capacity.

Rock grades were judged in reference to propagation velocity of elastic waves in many cases.

In case that river bed gravels or weathered rock are covering on the foundation, the in-situ shear strength can be taken as a criterion whether they are removed by further excavation or not in some cases.

For 9 dams, the excavation for filter covering was specified to be made as done for core covering. Since the filter covering must be simultaneously enhanced in impermeability in the portion in contact with the core, excavation standards as applied to core are surmised to have been applied practically.

The maximum excavation grade of the abutment was specified variously, ranging from 1:0.2 to 1:1. Five dams adopted 70° (1:0.36) or more, being mainly affected by topographic factors. On the other hands, for bed rocks deeply weathered, gentle grades were adopted.

For foundation of rock-fill zone, surface soil and heavily weathered and loose rocks are generally removed, and was excavated to such an extent as to secure sufficient bearing power and as sound enough not to cause differential settlement under heavy burden of dam body.

b) Treatment of unconsolidated sedimentary stratum

Taluses and river bed deposits judged to be equivalent or superior to the dam body material in strength and deformation resistance were left in many cases after examinations by in-situ tests and stability analysis. On the other hand, when they contained clay, they were removed because of adversely affection for the deformation resistance and hydraulic stability. The bed deposit layer was removed as far as possible in case of core covering, but when it could not be removed due to topographic conditions, excavation was adjusted in reference to density or Lu values, and in other cases soil blanket or grouting was applied.

(2) Shaping methods for contact surface

a) Method of finishing the contact surface of foundation rock.

Near the finishing surface of foundation rocks, the excavation by blasting was avoided not to loosen the rocks. In many dams, the contact surface was finished by manpower excavation by use of pick hammers in the section of 30 cm to 50 cm thickness.

The uneven rock surfaces which may disturb compaction effect, were made even by the methods such as (1) removal of overhanging face, (2) filling of recesses with concrete or mortar, (3) removal of detached blocks and sediments, and (4) cleaning of rock surface (by manpower or water jet) (See Fig. 1).

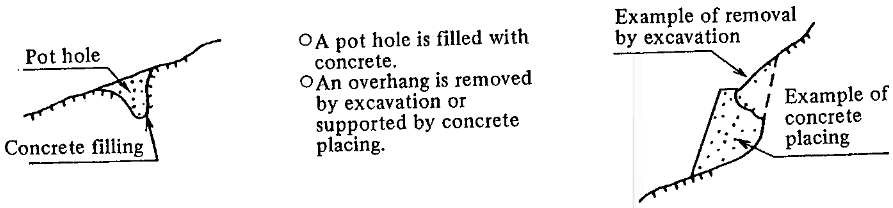


Fig. 1 Treatment examples of uneven rock surfaces

The contact face between concrete structures and core material such as inspection gallery surface or spillway wall was chipped by pick hammers in many cases. These methods of treatment were used to secure the adhesion between core materials and rocks, and to enhance tightness between core material and foundation rock to get resistant force against infiltration breaks.

b) Finishing of filter covering

The finishing of filter zone foundation in the section in contact with the core zone is surmised to be often made as done for the core zone. There were several cases where it was specified to finish a section exceeding 50% of the filter covering width as done for the core covering.

(3) Remedy treatments of faults, joints and open cracks

a) Replacement of faults by concrete and fill material

The faults in the contact surface of the foundation rocks, if very different from the surrounding rocks in deformability, cause the differential settlement in core zones, and eventually cracks.

And if these zones are highly permeable, they may cause the core to be eroded. The remedy treatment of faults has remained almost the same irrespective of these 30 years.

That is, in case for a seam of the contact surface rocks of the core zone foundation and for a fracture zone of less than 2 m in width, they were excavated and removed in a range a little wider than the weak portion, being replaced by concrete. And furthermore, when there was a problem in permeability, partiaclar grouting was made to treat the fault.

The faults of more than several meters in width distributed on core zone foundation, were replaced by cap concrete as surface treatment, and further grouting was applied.

In several cases, a weakly fractured zone extended on core covering along the flow direction, and in these cases, for filter and rock covering, the surface of the weak portion was covered with a soil blanket or filter material blanket for protection to seepage flow. The replacement by the core material was adapted when the rocks around the fault were soft tuffs, weathered rocks, etc.

In case the range of replacement was consequentry large, the construction method was decided by examining the stress concentration caused by concrete replacement and the deformability of core replaced, using stress deformation analysis.

b) Treatments of joints and open cracks

For the small cracks and seams appearing over the whole area of cotact surface rock, as a general trend, cement milk or morter was poured to close the openings and cavities, and the surface layer was carefully coated with paste.

This, including the low pressure shallow grouting, is called slush grouting. Slush grouting was widely used for weathered rocks, fracture zones, loose areas aroud fracture zones, and open cracks.

There was a case in which more than 1000 kg of cement was poured into one crack.

In the cases of slush grouting, the bore depth was less than 2 m, and bore intervals was between 1 m and 2 m. Morter or concrete was blown on rock surface to prevent leakage of slush grout and it was removed before embankment.

(4) Contact material

The contact material used till the middle of 1970s was generally a core material with the smaller maximum grain size. The idea of contact material clearly distinguished from core material was established in about 1975, and the use of especially selected loamy soil increased to attain the adhesion between rocks and the core zone.

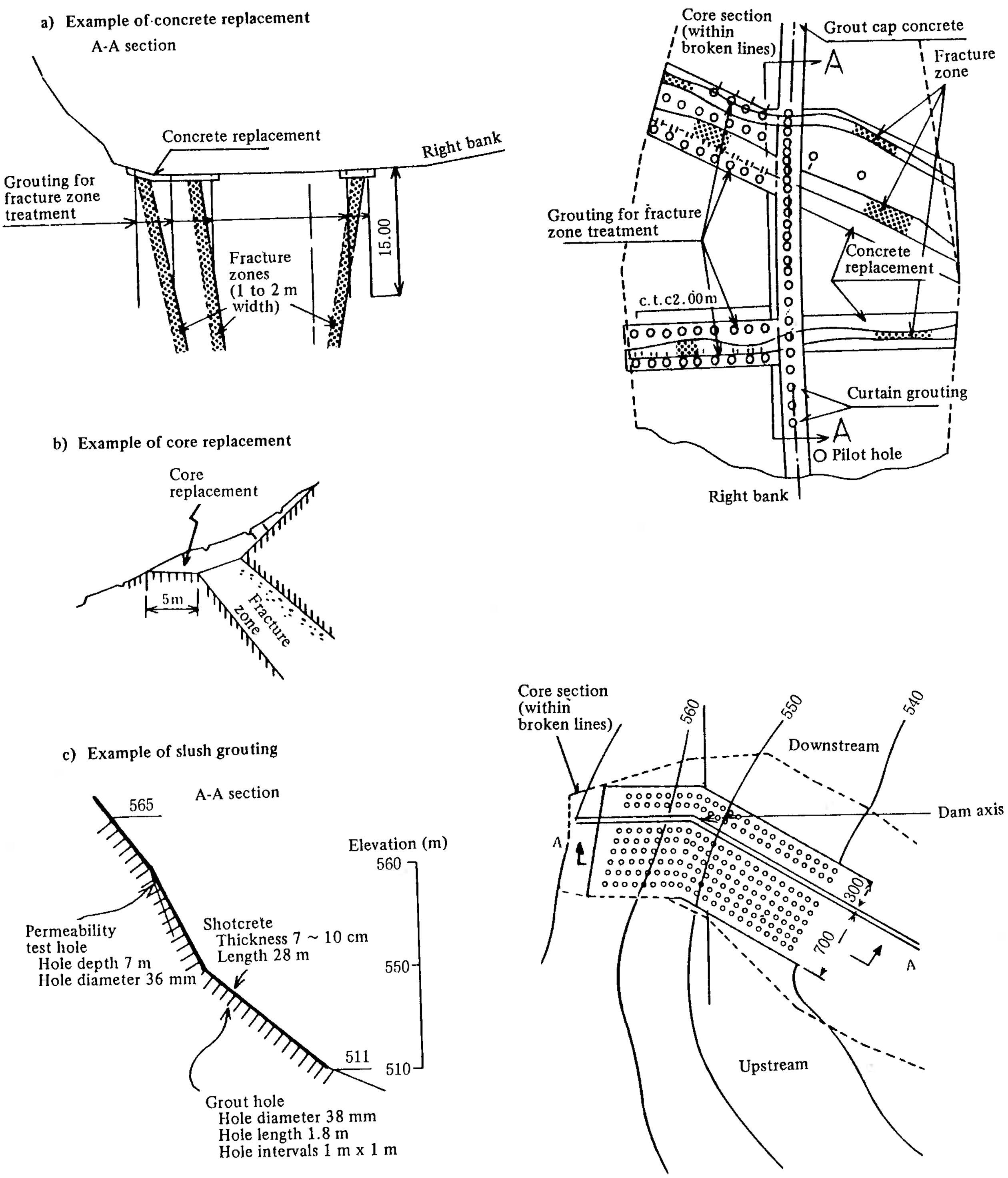


Fig. 2 Replacement by concrete or core materials, and slush grouting at the superficial area of fracture zones

The trend progressed further, and recently, contact clay (cohesive soil) and contact core (a little highly cohesive core material) is selectively used.

And in many cases, after water has sprinkled over the cleaned contact surface, slurry mud is coated before the contact material is applied.

The materials of contact clay range from 20 mm to 50 mm in maximum grain size, 40 to 60% in 74 μ or less grain content and 15 to 35 in plasticity index in most cases (See Fig. 3).

Dams completed between 1953 and 1974 (15 cases)

Kind of contact material	Same kind as core material (more humid than core material)	Cohesive soil	Nil (core material)
Maximum grain size (mm)	50 ~ 40	50~40	No specification
Number of cases (dams)	7	2	6

Applied thickness (cm)	50	30	20	10	5	5	3~5
Number of cases (dams)	1	1	1	1	2	1	1

$-74\ \mu$ content (%)	Less than 20	20 ~ 30	40 ~ 50	70
Number of cases (dams)	1	4	1	1
(Maximum grain size cm) Number of cases	(50)	(50) 1 (40) 3	(50)	(40)
Plastic index	7*	17* 14, 11, 3	9	21

* Values of core material

Dams completed between 1975 and 1982 (19 cases)

Kind of contact material	Same kind as core material (maximum grain size cut and more humid than core material)			Cohesive soil such as loamy soil				
Maximum grain size (mm)	50 ~ 40	40~20	10~5	60~50	25 ~ 20	15	10	2
Number of cases (dams)	4	2	2	3	4	1	1	2

Applied thickness (cm)	30	20	10	5 ~ 10	5	50	40	20	10	10~5
Number of cases (dams)	2	2	2	1	1	1	1	2	4	3

$-74\ \mu$ content (%)	Less than 20	20~30	30~35	44	25	40 ~ 60	60 or more
Number of cases (dams)	2	2	3	1	1	6	4
(Maximum grain size cm) Number of cases	(50) 1 (25) 1	(50) 2	(50) 1 (5) 2	(25)	(15)	(50~60) 3 (25~15) 2 (2) 1	(25) 1 (20) 1, (2) 1
Plastic index	20*~35 12~14	29 21	15	13	15	46, 34, 32 19, 13	24 23 14

Fig. 3 Properties of contact materials

3. Types and features of inspection galleries

(1) The purposes of installing inspection galleries can be arranged as follows,

i) Grouting is performed through the inspection gallery for portions not reliable enough in view of impermeability in foundation rocks.

ii) The dam body weight is utilized as a load for high pressure grouting, to enhance effect of grouting.

iii) The inspection gallery is used to observe the behavior of seepage flow and pore pressure, for safety control of the fill dam.

iv) When any abnormality is found in the impermeability of foundation rocks during or after ponding, re-grouting can be made through the inspection gallery.

v) As a side effect, the leads of instruments for observing the behavier of the dam body are collected in the inspection gallery, for safe instrumentation.

(2) The types of inspection galleries can be roughly classified into culvert system and tunnel system, depending on their constructed positions. Fig. 5 shows 16 dams adopting culvert system, 5 dams adopting tunnel system and 8 dams adopting both.

It can be seen that culvert system is adopted widely. The features of these systems are shown in Table 2, including merits and demerits respectively.

Typical standard sections of inspection galleries based on both the systems are shown in Fig. 4 as examples. As for inner dimensions of inspection galleries, those of 2.0 m bottom width × 2.5 m height seemed to be the standard and the others are not so greatly different from them dimensionally.

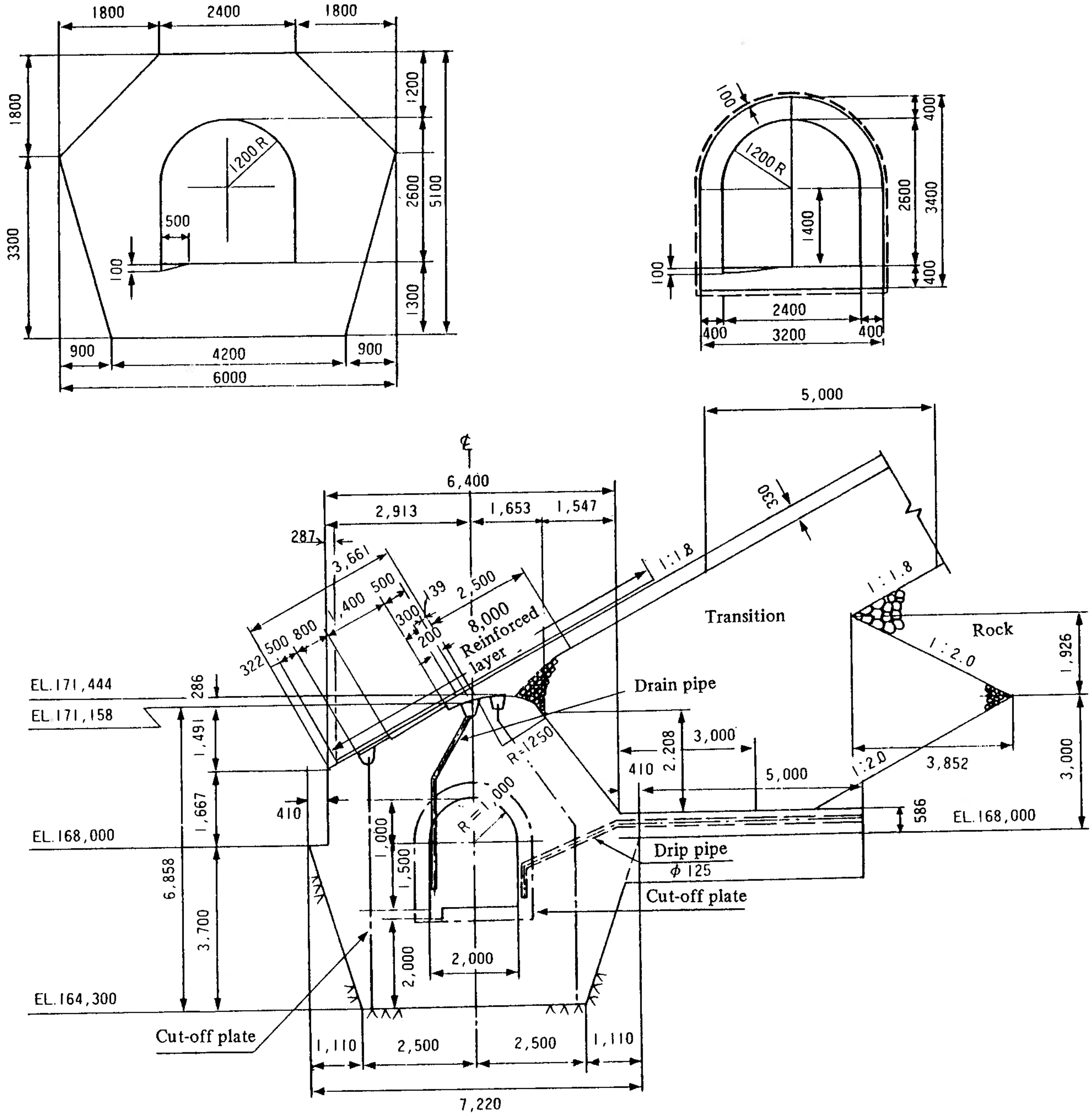
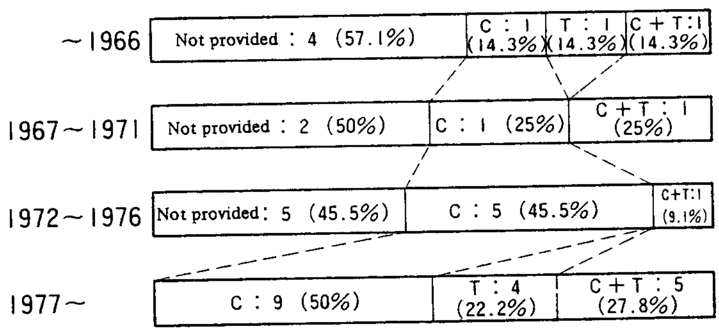


Fig. 4 Examples of standard cross section of respective inspection gallery types



C: Culvert type T: Tunnel type T + C: Both types used

Fig. 5 Changes for inspection gallery provision with the lapse of times

Table 2 Types and features of inspection galleries

Items	Culvert type (rock surface layer)	Tunnel type (deep in the rock)
1. Seepage flow intercepting effect	The effect is reliable since grouting can be executed directly at the rock surface portion. The inspection gallery itself has the effect of cutting off.	Grouting is made upward, viz. toward the rock surface. Since the positional relation between the rock surface and the tips of grout holes is unknown, whether perfect grouting has been done near the rock surface remains doubtful. Since upward grouting is made against gravity, reliability is not high enough.
2. Stability of rocks involved in excavation	The rock surface layers may be loosened, or their failling-in may occur. Excess excavation may happen in cross section.	The rocks around the tunnel may be loosened, but the rock surface layer cannot be loosened.
3. Influence of dam weight on foundation rocks	The rock surface layer is displaced and deformed. When the gallery passes through a weak layer such as a fracture zone, differential displacement occurs at the boundary. In case of soft rocks, tensile strain and cracking occur in high portions.	Displacement and deformation are slight in the depth of the rock.
4. Influence on inspection gallery itself	Since the difference in rigidity between concrete and rocks is large, the stress is differently shared. Strain which exceeds the allowable strain (compressive, tensile and shearing) of concrete may be caused.	Since the strain caused is small even if there is difference in rigidity, the difference in stress sharing is small. Strain which exceeds the allowable strain of concrete is not occurred.
5. Design of inspection gallery.	Design must be made according to the change in the cross section of inspection gallery, and the inrervals of joints in the longitudinal direction must be examined.	Any special consideration is not required.
6. Influence of inspection gallery on stresses and deformations in the core zone	If the inspection gallery protrudes above the rock surface, the differential settlement in core may occure, and therefore the core must be carefully designed and constructed.	No influence
7. Relation with construction period	The construction of inspection gallery affects the construction speed of the core.	The construction of inspection gallery can be made irrespective of the construction of the core.

4. Past provision of inspection galleries and their relation with geological features

(1) The past provision of inspection galleries with the lapse of years is shown in Fig. 5. According to it, before 1976, dams with no inspection gallery provided were more than those with it provided, and even if provided, the emphasis seems to have been placed on the treatments of local defects existing in the foundation rocks (See Table 3).

Table 3 Examples of geological features and inspection gallery provision

Year	Not provided	Provided		
		River bed. section	Abutment section	Entire foundation
~1966	■ ○ □ □	△ (Fault portion)	■ (Left bank)	(×)
1967~1971	□ ○			(●) ○
1972~1976	△ ○ ○ ● ●	▲	○ (Right bank) ○ ● (Left bank)	(△) (■) ○
1977~		● (River bed to middle stage of left bank) ● (River bed to right bank)	□ (Left and right banks) △ (Left and right banks)	⊙ × ■ × ▲ × △ × × × ⊙ × ○ ×

However in June, 1976, failure of Teton Dam in USA has occurred, and it was pointed out that the failure was caused by the infiltration breakage of the core at the interface portion. Since then, the necessity of sufficient foundation treatment has been discussed for construction of safer fill dams, and the provision of the inspection gallery has been executed for all dams. Fig. 5 shows the history graphically.

(2) The provision of inspection galleries has close relation to geological features. The conditions of the provision for the respective age groups are shown in Table 3, with the kinds of rocks as the parameter. From the table, it can be seen that before 1976, inspection galleries were provided to cover local defects in hard rocks. For facing type dams, the inspection gallery was provided for the entire range of the foundation at their toe section, because of their characteristics. These conditions are surmised to have continued till 1976 on the same conception.

However, it became difficult to obtain geologically good dam sites, and areas of geologically younger rocks came to be selected as dam sites. In this situation, the inspection gallery became necessary to attain filtration control satisfactorily by foundation treatment. Under such circumstances, the failure of Teton Dam occurred.

Therefore, in and after 1977, as shown in Table 3, while the fill dams founded on relatively hard rocks had the inspection gallery on river bed or on abutment, many other sites had the inspection gallery provided to cover the entire section of the foundation.

Table 4 Geological features and inspection gallery types

Year	Culvert type	Tunnel type	Both types used
~1966	(×)	■	△
1967~1971	(●)		○
1972~1976	(△) △ (■) ○ ○		●
1977~	△ ⊙ □ △ × × ⊙ ○ ×	× ■ ● ●	× ▲ × × ×

Legend ( ) Facing type

- ○ Sedimentary rocks of Palaeozoic to Mesozoic
- ⊙ Crystalline schists
- ● Granites
- ▲ Gneisses
- △ Acidic volcanic rocks of Mesozoic to Palaeogene
- □ Sedimentary rocks of Mesozoic to Palaeogene
- × Sedimentary rocks of Neogene
- ■ Volcanic rocks of Neogene

Table 4 shows the relation between geological conditions and types of inspection galleries. From the table, it can be seen that culvert system is adopted predominantly. After 1977, there were 9 cases of culvert type, 4 cases of tunnel type and 5 cases using both types. The kinds of rocks at the sites where culvert type is adopted include 1 case of sedimentary rocks of Palaeozoic to Mesozoic, 2 cases of crystalline schists, 2 cases of acidic volcanic rocks of Mesozoic to Palaeogene, 1 case of sedimentary rocks of Mesozoic to Palaeogene and 3 cases of sedimentary rocks of Neogene, indicating that there is no relation between the type of inspection galleries and kinds of rocks. Out of 4 cases adopting tunnel type, 2 cases were on granites, and the remaining 2 cases were on sedimentary rocks and volcanic rocks of Neogene. In the latter cases, the geological features are surmised to have greatly affected in selection of the inspection gallery type. Out of 5 cases using both the types, 4 cases were mainly on the sedimentary rocks of Neogene, and it can be presumed that culvert type was adopted in relatively hard portions, and that tunnel type was adopted in weathered and altered rock.

The results of the investigation have been summarized above. As to what type of inspection gallery should be provided at a site of specific geological conditions, it is surmised to be very important to examine the respective items as shown in Table 2 based on past experiences, for making proper judgement.

Chronological Table

Year	Dam Constructions (Dam Height, Location) and Related Remarks	Theories, Standards and Guidelines
BC36	Yosami-ike Reservoir	
162	Kaerumata-ike Reservoir (17 m, Nara)	
466	Ichiban-ike Reservoir (15 m, Ehime)	
708	Sumiyoshi-ike Reservoir (20 m, Kagoshima)	
700s	Mannoh-ike Reservoir (23.5 m, Kagawa); raised to 32 m height and used still now.	
1128	Daimon-ike Reservoir (32 m, Nara)	
1913	Ohno Dam (37 m, Yamanashi)	
1923	Murayama-upper Dam (24 m, Tokyo) constructed in June, and hit by the Kanto Earthquake in September. Slight movement of concrete facing, 18 cm settlement on creset and several cracks observed.	
1925		The seismic coefficient method for dam design proposed by Dr. N. Mononobe.
1927	Murayama-lower Dam (30 m, Tokyo)	
1934	Yamaguchi Reservoir (33 m, Saitama)	
1938	Shohwa-ike Reservoir (35 m, Okayama)	
1953	Ishibuchi Dam* (53 m, Iwate); the first reinforced concrete facing rockfill dam in Japan.	
1954	Nozori Dam (44 m, Gunma), and Sannohkai Dam (37 m, Iwate)	
1955	Nishigoh Dam (33 m, Fukushima)	
1956	Hatori Dam (37 m, Fukushima)	
1957		The Design Criteria for Dams established by Japanese National Committee on Large Dams (JANCOLD).
1960	Miboro Dam* (131 m, Gifu) constructed with a great contribution to such as use of large construction machineries and quality control methods of embankment materials.	
1961	Makio Dam* (105 m, Nagano)	
1963	Minase Dam* (65 m, Akita), and Ohshirakawa Dam* (95 m, Gifu)	
1965	Honzawa Dam* (73 m, Kanagawa), and Yanase Dam* (115 m, Kochi)	
1967	Kuzuryuh Dam* (128 m, Fukui)	
1968	Misakubo Dam* (105 m, Shizuoka), and Ohtsumata Dam* (52 m, Fukushima); the first asphalt-concrete facing rockfill dam in Japan.	
1969	Kisen-yama Dam* (91 m, Kyoto)	
1971		The first revision of the Design Criteria for Dams completed.
1972	Shimokotori Dam* (119 m, Gifu), and Tataragi Dam* (65 m, Hyogo)	A Guideline for Grouting of Dam-foundation Rock established by Japan Society of Civil Engineers (JSCE).
1973	Hirose Dam* (75 m, Yamanashi), Kurokawa Dam* (98 m, Hyogo), Niikappu Dam* (103 m, Hokkaido), and Uchitani Dam* (64 m, Kumamoto)	Case Histories of Grouting of Dam-foundation Rock published by JSCE.
1974	Aburatani Dam* (82 m, Kumamoto), Daisetsu Dam* (87 m, Hokkaido), Miyama Dam* (75 m, Tochigi), and Mizukubo Dam* (62 m, Yamagata)	Tentative Guideline for Chemical Grouting enforced by Ministry of Construction.
1975	Iwaya Dam* (128 m, Gifu), Myohjin Dam* (89 m, Hiroshima), and Nabara Dam* (86 m, Hiroshima)	
1976	Futai Dam* (87 m, Niigata), and Kassa Dam* (90 m, Niigata)	Standards for Geological Investigation of Dam's Foundation established by JANCOLD. In-situ Plate Load Test for Rock Deformation standardized by JSCE.
1977	Miho Dam* (95 m, Kanagawa), Nanakura Dam* (125 m, Nagano), Seto Dam* (111 m, Nara), Takase Dam* (176 m, Nagano); the highest rockfill dam in Japan, and Terauchi Dam* (83 m, Fukuoka)	Geological Survey for Dam published by JSCE. Standard of Lugeon Test (Draft) promulgated by Ministry of Construction.
1978	Shirakawa Dam* (66 m, Yamagata), and Tedorigawa Dam* (153 m, Ishikawa)	The second revision of the Design Criteria for Dams completed by JANCOLD. In-situ Shear Test of Rock – Guidelines and Commentaries – published by JSCE.
1979	Futaba Dam* (60 m, Hokkaido), Isarigawa Dam* (42 m, Hokkaido), and Muri Dam* (16 m, Hokkaido); the first asphalt-concrete core rockfill dam in Japan	
1980	Gosho Dam* (53 m, Iwate), and Usogawa Dam* (56 m, Shiga)	
1981	Inamura Dam* (92 m, Kochi) and Tamahara Dam* (116 m, Gunma)	
1982	Oh-uchi Dam* (102 m, Fukushima), Takami Dam* (120 m, Hokkaido), and Tokachi Dam* (84 m, Hokkaido)	

Notes: 1) The year of construction indicates the year of completion of embankment work.
2) * indicates dams studied by the present questionnaires.

Lists of Working Group Members

Chairman	KIKUCHI, Kohkichi	Engineering Geological Department, Tokyo Electric Power Service Co., Ltd.
Manager	NAGAYAMA, Isao	Public Works Research Institute, Ministry of Construction
	HIRAI, Akira	Construction Department, Tokyo Electric Power Co., Inc.
	NISHIWAKI, Yoshifumi	Construction Department, Tokyo Electric Power Co., Inc.
Leader of Working Group	MATUI, Ietaka	Civil Engineering Laboratory, Central Research Institute of Electric Power Industry
Sub-leader of Working Group	OHMACHI, Tatsuo	Graduate School at Nagatsuta, Tokyo Institute of Technology
Member	IKEDA, Takashi	Public Works Research Institute, Ministry of Construction
	IWANAGA, Tateo	Construction Department, Kansai Electric Power Co., Inc.
	OKABAYASHI, Nobuyuki	Kajima Institute of Construction Technology, Kajima Corporation
	KAKUTA, Takahiko	Civil Engineering Laboratory, Central Research Institute of Electric Power Industry
	KAWASHIMA, Yukio	Civil Engineering Department, Electric Power Development Co., Ltd.
	KAZURAYAMA, Fumiharu	Civil Engineering Department, Electric Power Development Co., Ltd.
	KUWAHARA, Keizo	Public Works Research Institute, Ministry of Construction
	SADAHIRO, Takeyoshi	Research Institute, Water Resources Development Public Corp.
	SHIMIZU, Kunio	Geological Department, Nippon Koei Co., Ltd.

Member	TAKAGI, Hajime	Civil Engineering Division, Technology Department, Shimizu Construction Co., Ltd.
	TAKEOKA, Kenjirou	Construction Department, Kansai Electric Power Co., Inc.
	NAKAGAWA, Takeshi	Civil Engineering Division, Engineering Headquarters, Ohbayashi-Gumi, Ltd.
	HIRANO, Isamu	Public Works Research Institute, Ministry of Construction
	FUJIEDA, Makoto	Engineering Geological Development Tokyo Electric Power Service Co., Inc.
	FURUBE, Hiroshi	Engineering Department, Hazama-Gumi, Ltd.
	MIZUNO, Mitsuaki	Chubu Regional Construction Bureau, Ministry of Construction
	MOMIKURA, Katsumiki	Land Bureaus, National Land Agency
Ex-member	ISHII, Kiyoshi	Formerly of Construction Department, Tokyo Electric Power Co., Inc.
	IWASAKI, Takashi	Formely of Construction Department, Kansai Electric Power Co., Inc.
	KUSUNOKI, Ken-ichirou	Formerly of Civil Engineering Laboratory, Central Research Institute of Electric Power Industry
	KURODA, Shigenori	Formerly of Civil Engineering Department, Electric Power Development Co., Ltd.
	KONDO, Satoru	Formely of Public Works Research Institute, Ministry of Construction
Writer, Non-member	KUDO, Hiroshi	Agriculture Structure Improvement Bureaus, Ministry of Agriculture, Forestry and Fishery

THE JAPAN SOCIETY OF CIVIL ENGINEERS
YOTSUYA 1-CHOME, SHINJUKU-KU
TOKYO, 160 JAPAN