

Behaviors of Rockfill Dams and Safety Monitoring during First Filling of the Reservoirs in Japan

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Public Works Research Institute
Ministry of Construction
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Preface

The objectives of this research work are to investigate the behavior of rockfill dams during first filling of the reservoirs and to develop more effective instrumentation for the safety monitoring of rockfill dams.

We have accumulated a lot of experience in concrete dam engineering in Japan and published a manual for safety control of concrete dams. We, however, have not determined all the parameters pertinent to the safety control of rockfill dams. The stress-strain and seepage properties of rockfill dams are more complicated than concrete dams. For example, while the stress-strain properties of rockfill dams are inelastic and dependent on the type of materials and method of construction, the stress-strain properties of concrete dams are elastic and the displacement of the dam subjected to water pressure and other loadings can be calculated and the limits of displacements can be defined.

The safety control during the period of the first filling of the reservoirs is particularly important, because failures of dams have frequently occurred during this period.

The rockfill dams reported here are zoned type dams which have rather thin impervious core zones and wide rockfill zones. The height of the dams ranges from 60 m (197') to 131 m (430').

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1. Leakage in Rockfill Dams

1-1. Measurements of Leakage Using the Weir at the Downstream End of the Dam

The weir is often set up at the downstream end of the dam to measure the leakage in rockfill dams as shown in Fig. 1. This system is easily used with dams where the valley or creek of the site is U or V shaped and there is a small amount of river bed deposit.

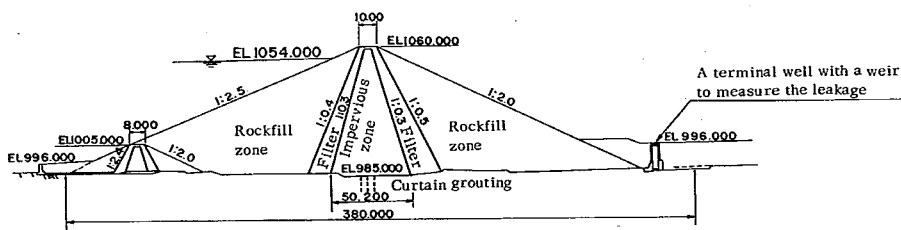


Fig. 1. A weir to measure leakage at the downstream end of the dam.

The discharge measured using this system includes

1. Leakage through dam body
2. Leakage through foundation
3. Ground water through abutments
4. Surface water on each abutment
5. Precipitation on the downstream slope of the dam.

The discharge measured at the weir as shown in Fig. 1 is not always the whole leakage of the dam. It is not possible and not always necessary to measure the whole leakage in rockfill dams accurately. As the discharge at the weir should be a useful index of the whole leakage, it will be described simply as "leakage" hereafter.

Fig. 2 shows the leakage and precipitation at the dam site and the water level of the reservoir in A Dam during the eight months after the initiation of impounding. The impounding was initiated in February. When the water level of the reservoir went up to 27 m (89') in the beginning of April, the leakage began to

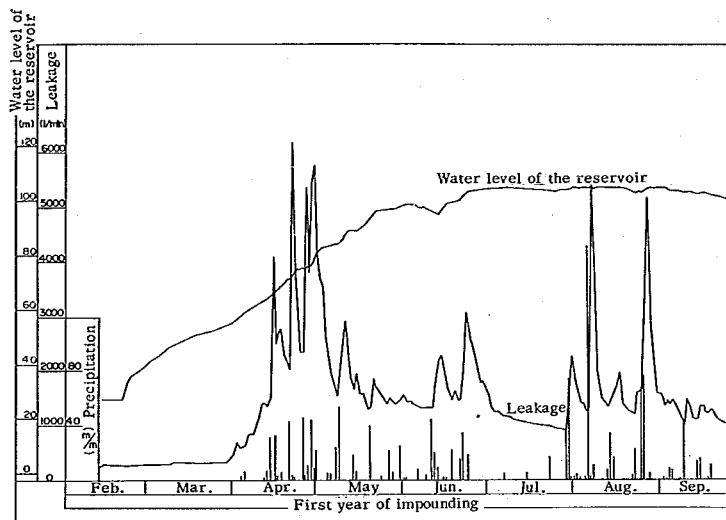


Fig. 2. Leakage, water level and precipitation in A Dam.

increase rapidly. While the water level was 30 m (98') to 40 m (131') the leakage varied considerably in quantity from 2,000 l/min (530 gallon/min) to 6,000 l/min (1,600 gallon/min). It should be concluded that the rapid increase of leakage is due to the discharge from factors 4 and 5 stated above. While the water level continued to rise during June, the leakage decreased slightly. When the reservoir is filled full with water in July, the leakage decreases gradually because of little rain in this month. Leakage amounts increased to 5,000 l/min (1,300 gallon/min) temporarily in August and September because of heavy rain. Leakage of A Dam is about 1,000 l/min (260 gallon/min) in July when there is little rain, and there was 200 l/min (50 gallon/min) of "leakage" before the initiation of impounding. Accordingly it would be concluded that the leakage through dam body (discharge 1 stated above) would be approximately 800 l/min (210 gallon/min).

Fig. 3 shows the leakage, precipitation and the water level of the reservoir in B Dam for three years. The leakage varies much from 300 l/min (80 gallon/min) to 7,000 l/min (1,850 gallon/min). But most of the leakage over 500 l/min (130 gallon/min) would be surface water due to rainfall. Rainfall is smaller in the

winter (Nov. - Mar.) than any other seasons. While the water level drops from 95 m (312') to 65 m (213') in the winter, the corresponding leakage drops from 350 l/min (92 gallon/min) to 150 l/min (40 gallon/min). Therefore 200 l/min (53 gallon/min), which is the difference between 350 l/min and 150 l/min, is the net leakage through the dam body and foundation. Though Fig. 3 shows the behavior of only three years, the leakage stabilized at from 150 l/min (40 gallon/min) to 200 l/min (53 gallon/min) six years later.

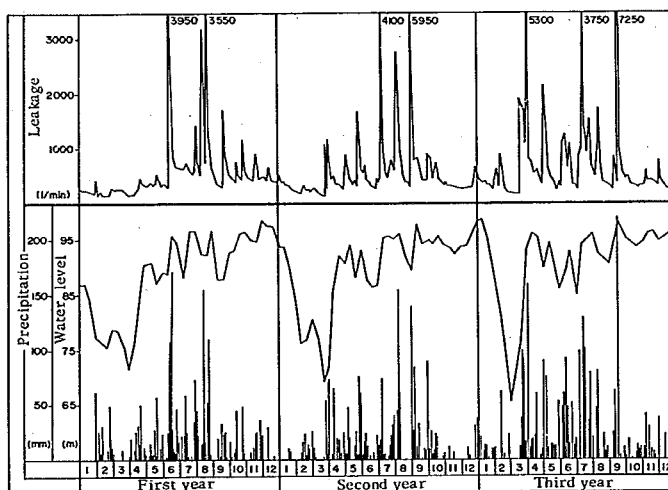


Fig. 3. Leakage, water level and precipitation in B Dam.

The similar results are shown in Fig. 4. The leakage in Fig. 4 is also under the influence of precipitation, and it decreases much in the winter, because the water level is low and the surface water is frozen in this season.

Figs. 2 to 4 show that it is difficult to measure leakage in good accuracy using the weir system as shown in Fig. 1 which is easily influenced by surface water or precipitation. However the net leakage without the influence of rainfall can be measured fairly well if the inflow of the surface water is small. Fig. 5 shows the water level and leakage relation for the first 14 months in D Dam. While the water level goes up to the elevation of 50 m in the first two months, the

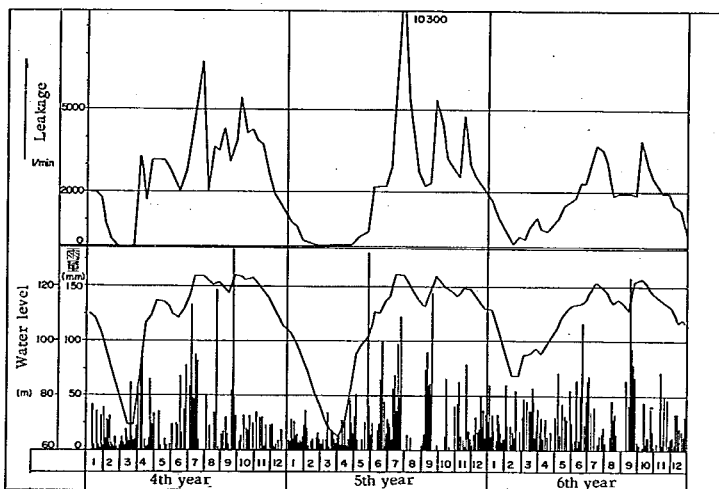


Fig. 4. Leakage, water level and precipitation in C Dam.

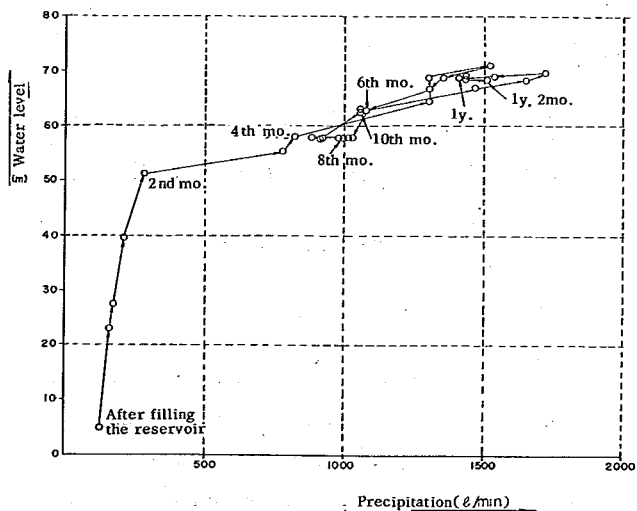


Fig. 5. Relationship between leakage and water level in D Dam.

tion. But we cannot know the individual locations and quantities of the leakage flow. Accordingly we have developed instrumentation and procedures to measure leakage

leakage is nearly proportional to the water level and the maximum leakage is 300 l/min (80 gallon/min). While the water level goes up to the elevation 70 m from 50 m, the leakage rate increases considerably and another proportional relation will be found between water level and leakage. The sudden increase of leakage above the water level of 50 m would be attributed to the seepage through the weathered granite rock of the left abutment of the dam site. The left abutment had been treated for its high permeability before the construction of the dam. This dam should be carefully controlled and the measurement of the leakage should be continued, since the leakage is large.

1-2. Refined Instrumentation and Procedures to Measure Leakage in Rockfill Dams

The measurements of leakage stated in 1-1 are easily affected by the discharge of the surface water. In case of Fig. 5 which is only slightly influenced by surface water, we might obtain the total leakage through the dam body and founda-

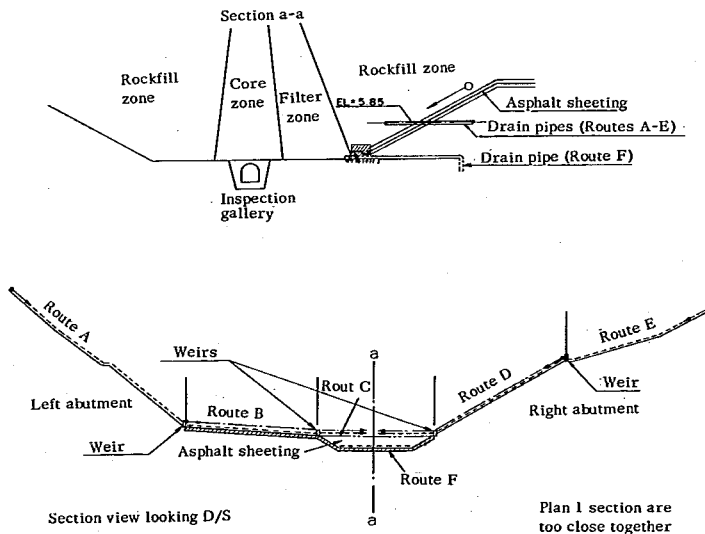


Fig. 6. Installations to measure leakage in E Dam.

in rockfill dams.

Fig. 6 shows a seepage collection system which determines seepage flows from five seepage areas. After the five seepage flows are corrected, they are drained through pipes to the terminal well which is located at the downstream toe of the dam. The discharge of each drain pipe is measured at the terminal well.

Satisfactory results were obtained in E Dam using the system as shown in Fig. 6. Fig. 7 shows the six independent leakages and water level of the reservoir. The maximum leakage is 10 to 20 l/min (2.6 to 5.3 gallon/min) in the A, B, C and D routes, and 80 l/min (21 gallon/min) in the F route which is the lowest elevation of the riverbed. Each leakage and the total leakage of A, B, C, D, E and F routes

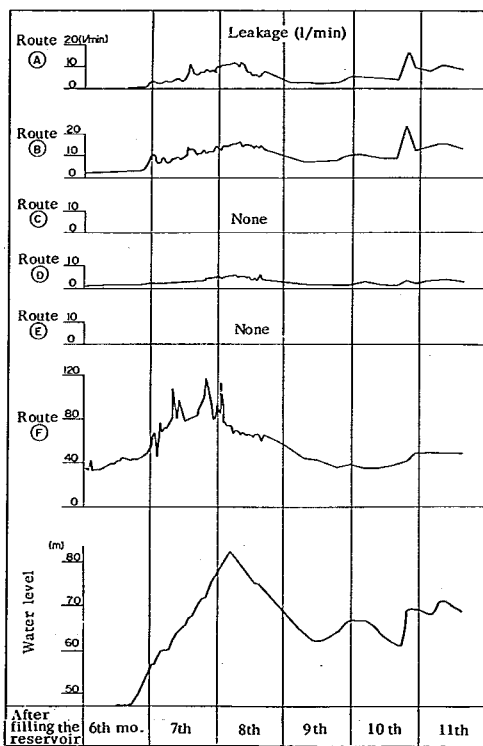


Fig. 7. Leakage in each route in E Dam.

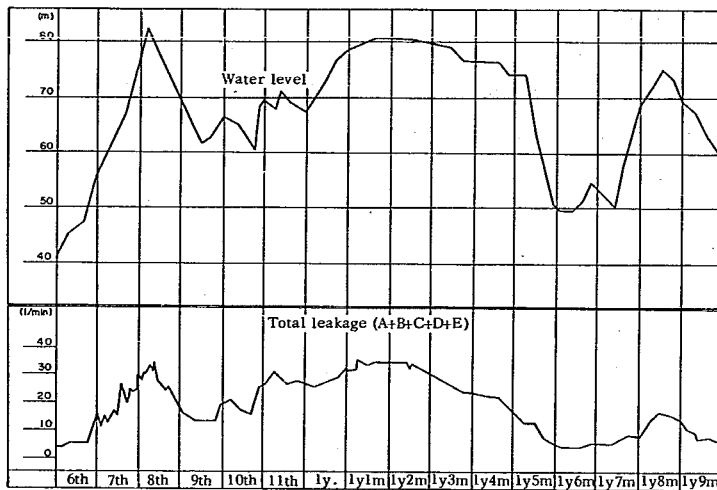


Fig. 8. Total leakage in E Dam.

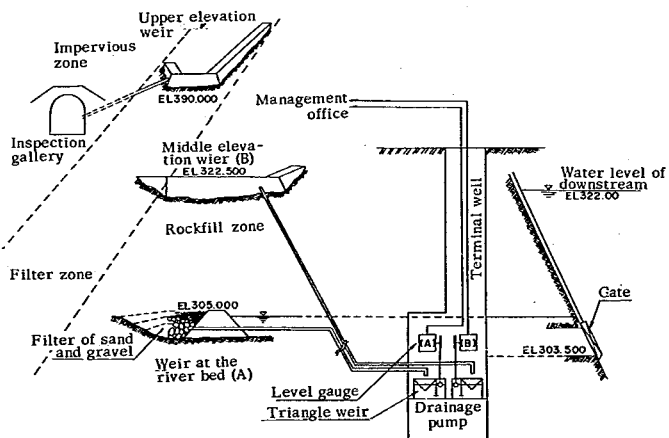


Fig. 9. Installations to measure leakage in F Dam.

correspond closely to the water level of the reservoir, and are not affected by the precipitation as shown in Fig. 7 and Fig. 8. Therefore, this system can be said to be an advanced one to measure the leakage in rockfill dams.

The instrumentation in Fig. 1 and Fig. 6 can be used for dams whose downstream water level is rather low or dams which have inspection galleries along the perimeter of the foundation. If the downstream water level is high and there is not an inspection gallery, another device should be supplemented to the system as shown in Fig. 6 in order to measure the leakage of the rockfill dams.

F Dam has a reregulating reservoir in its downstream area, and has about a 20 m (66') high downstream water level (Fig. 9). Two inspection galleries are located in each abutment. The leakage water collected at the upper intake weirs flows down to the inspection galleries through drain pipes and the quantity of the leakage is measured there. On the other hand, the leakage water collected at the middle and lower intake weirs is drained through pipes to the terminal well at the downstream toe of the dam, since the inspection gallery is located at a higher elevation. The terminal well is usually below the downstream water level. The

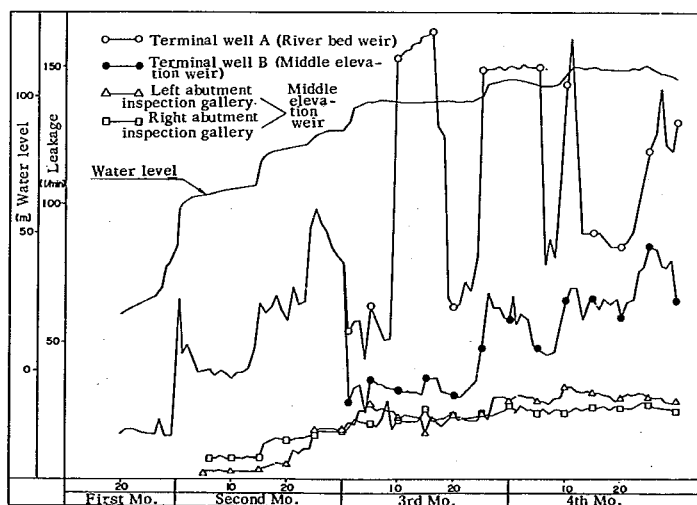


Fig. 10. Leakage in F Dam.

water level in the terminal well is equal to the downstream water level, because the terminal well cannot be completely sealed. The water in the terminal well is drained with drainage pump, when the leakage of middle and lower elevation is measured. Fig. 10 shows the leakage and water level of F Dam measured using the system as shown in Fig. 9. The leakages from upper elevations in the left and right abutments, which are drained to the inspection galleries, are approximately the same and proportional to the water level. The maximum discharge is about 30 l/min (8 gallon/min). The leakage of middle elevation (B) is also proportional to the water level and the maximum discharge is 70 l/min (18 gallon/min). On the other hand, the leakage at the river bed weir (A) increases remarkably when the downstream water level is high. This shows that water of the reregulating reservoir overflows the weir at the river bed (A), when the downstream water level is high. The leakage is also almost proportional to the water level of the reservoir, when the downstream water level is low, and maximum is about 100 l/min (26 gallon/min).

1-3. Measurements of Seepage Flow in the Foundation

The leakage described in 1-1 and 1-2 is mainly concerned with the leakage through the dam. The leakage through the dam is important of course, but the leakage through the foundation is much more important. The use of impervious materials and the construction of impervious zones have progressed in Japan such that large amounts of leakage would seldom happen. The rock foundation, however, sometimes have high permeability and the effects of grouting are difficult to confirm.

The seepage flow in the rock foundations can be observed using pore water pressure gauges, piezometers, and the leakage in inspection galleries and other service tunnels. From the point of view of the safety control of rockfill dams, the quantity of the seepage flow in rock foundations is more important than the pressure of the seepage flow.

The seepage flow in the foundations will be classified into two categories. One is the seepage flow in the shallow depths of the foundation and the other is deep. The former will be discussed here, since it is more important than the latter in the

safety control of dams.

Drain holes are usually located just downstream of the grout curtain in concrete gravity dams. Pore pressures and leakages are measured during the filling of the reservoir and the effects of grouting are confirmed. Though it is not necessary to reduce the uplift in rockfill dams, it would be useful to have observation holes, similar to the drain holes of concrete gravity dams, in the inspection galleries in the rock foundation of rockfill dams because the seepage flow could be measured. If we measure the leakage through those observation holes during the first filling of the reservoir, we can evaluate the effects of grouting. If the leakage is larger than expected, we should regROUT and improve the permeability of the foundation using inspection galleries. These procedures will greatly improve the safety of dams.

Fig. 11 shows the example of seepage observation holes which are located downstream of the grout curtain. The observation hole in Fig. 11 has a pressure gauge and valve at the end. The observation hole should be protected with casing and filter against erosion, if necessary.

Fig. 12 shows the water level of the reservoir, the leakage, and the pressure of the water in observation holes No. 5-1 and No. 5-2. The leakage and the pressure are proportional to the water level. The pressure of No. 5-2 hole is higher than No. 5-1 hole, whereas the leakage of No. 5-2 hole is smaller than No. 5-1. This shows that it is necessary to measure not only the pressure but also the quantity of the leakage.

Fig. 13 shows the results of observation hole No. 2-1. When the water level goes up to the elevation of 80 m (262') six months after the initiation of the impounding, the leakage of No. 2-1 hole amounts to 70 l/min (18 gallon/min). Judging from experiences with concrete dams the leakage under 100 l/min (26 gallon/min) per hole is not dangerous. But the water level of the reservoir was lowered to insure the safety, and supplemental consolidation grouting was performed. The regROUTing was successful. The leakage decreased to 2 l/min (0.5 gallon/min), which shows the suppression of the seepage flow around No. 2-1 hole. The pore water pressure of No. 2-1 hole was 4.0 kg/cm² (57 psi) before regROUTing and 4.5 kg/cm² (64 psi) after regROUTing. The passages of the seepage flow in the rock foundation were

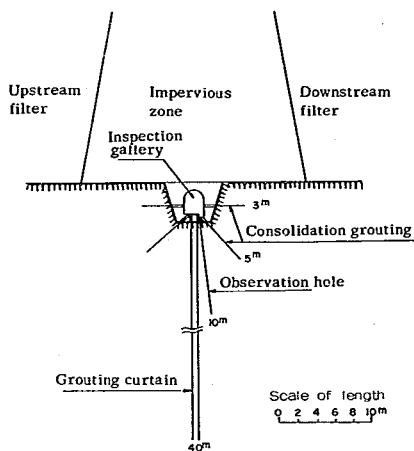


Fig. 11. Observation hole to measure the seepage flow in the foundation.

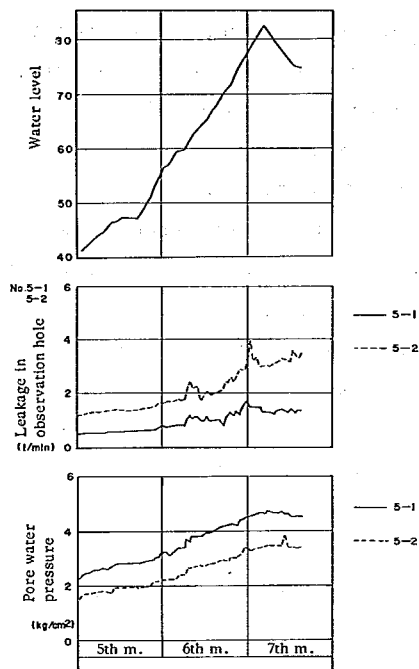


Fig. 12. Leakage and pore water pressure of observation holes (No. 5-1, 2) in E Dam.

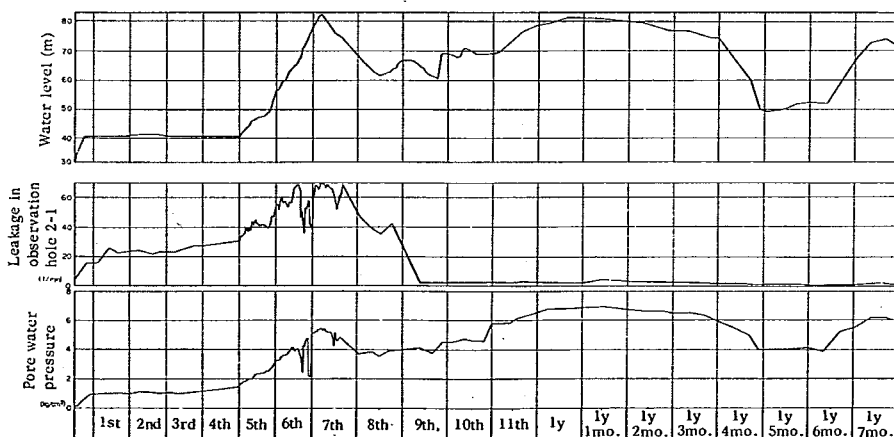


Fig. 13. Leakage and pore water pressure of observation hole (No. 2-1) in E Dam.

reduced by regrouting. As a result, the quantity of the leakage decreased, but the pore water pressure increased on the contrary. This shows that we cannot guess the quantity of leakage by measuring the water pressure.

Next, another example of the leakage in diversion tunnel and inspection galleries will be shown.

Fig. 14 shows the plan (downstream only) of G Dam. G Dam has a diversion tunnel in the left abutment and two access tunnels for grouting in the right abutment.

The leakage in these tunnels are shown in Fig. 15. The leakage in the diversion tunnel is due to the imperfect plugging of the tunnel and its maximum discharge is about 400 l/min (106 gallon/min) when the reservoir is full. The maximum leakage of upper access tunnel is about 200 l/min (53 gallon/min). The leakages in both of diversion tunnel and upper access tunnel are proportional to the water level of the reservoir.

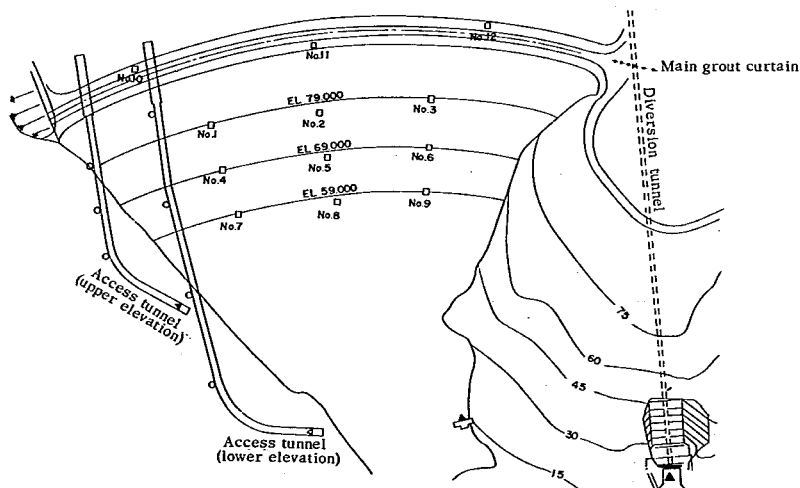


Fig. 14. Diversion and access tunnels in G Dam.

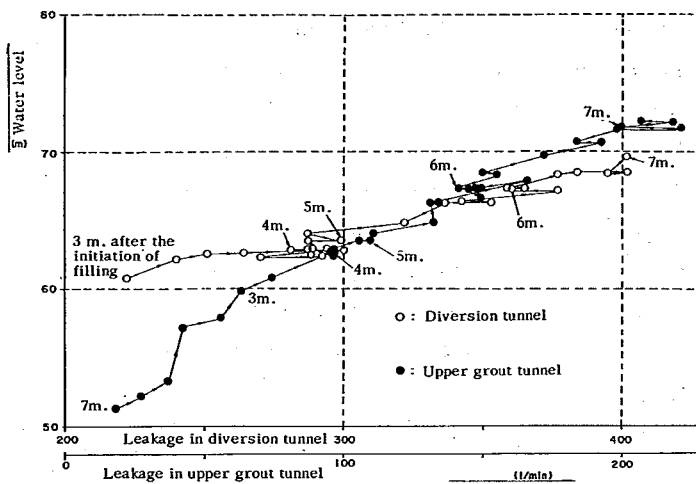


Fig. 15. Relationship between leakage and water level in G Dam.

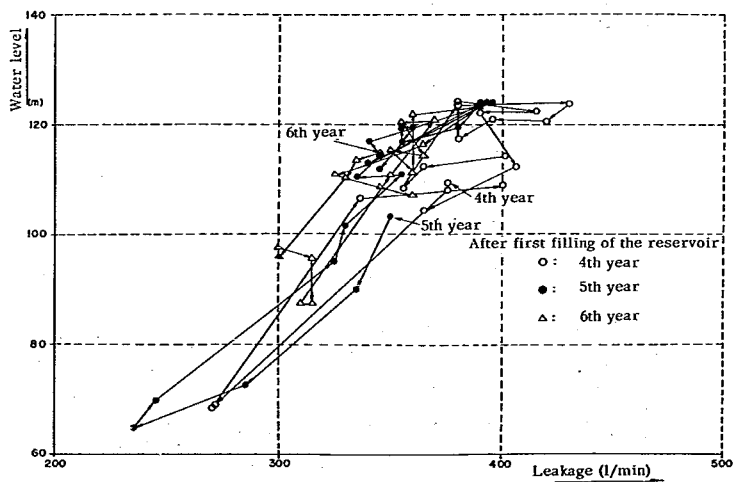


Fig. 16. Relation between leakage and water level in C Dam.

Fig. 16 shows the relationship between the total quantity of the leakage in the inspection gallery and the water level of the B Dam reservoir. The leakage in the gallery here means the leakage through joints and cracks of the concrete wall of the gallery and does not mean the leakage of observation hole as shown in Fig. 11.

Fig. 16 shows that the leakage is approximately proportional to the water level and gradually decreases year by year. The passage of seepage flow in the foundation will be closed with small particles or silts.

At the end of this paragraph, the leakage in the depths of the foundation will be shown. Fig. 17 shows the leakage in the depths of the shaft in the rock foundation of B Dam. The shaft was set up to measure the effects of fault dental works. The leakage in Fig. 17 has a slight relationship with the water level of the reservoir. This is the characteristic of the leakage in the depths of foundation. That kind of leakage does not affect the safety of dams. As the leakage in the foundations decreases little by little with time, as shown in Figs. 16 and 17, the stability of the foundation relative to seepage will increase.

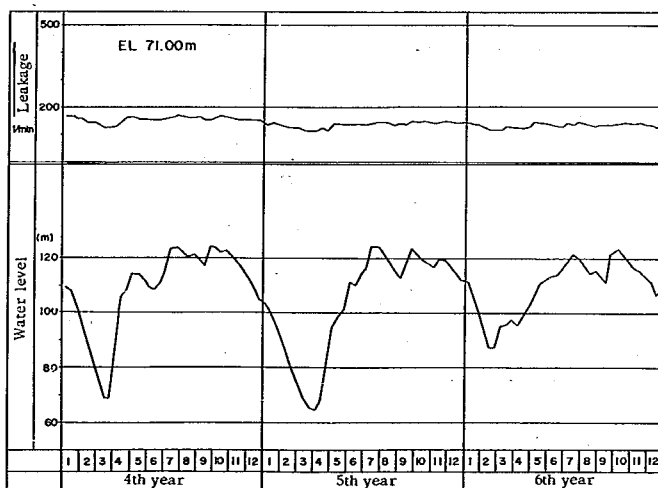


Fig. 17. Leakage in the depths of foundation in C Dam.

1-4. The Leakage in a Rockfill Dam with Concrete Facing Slab

Fig. 18 shows the maximum section of H Dam which is a rockfill dam with concrete facing slab. The leakage has been measured using a weir similar to that shown in Fig. 1, so the leakage includes both through the concrete slab and the foundation. Fig. 19 shows the leakage in winter and summer during the 12 years following the completion of the dam. (The normal high water level of the reservoir

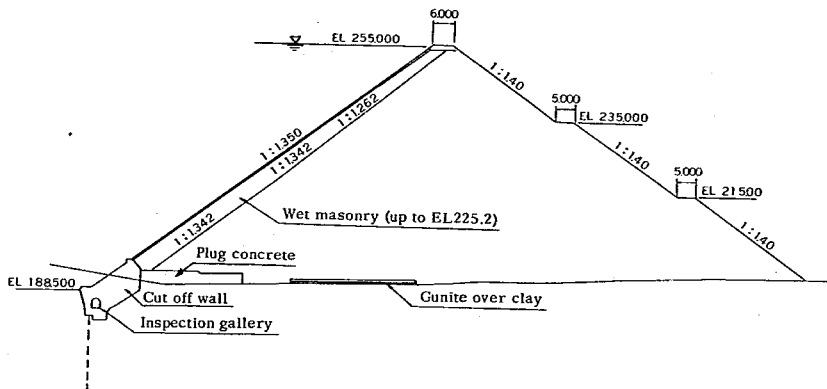


Fig. 18. Maximum section of H Dam
(Rockfill dam with concrete facing slab)

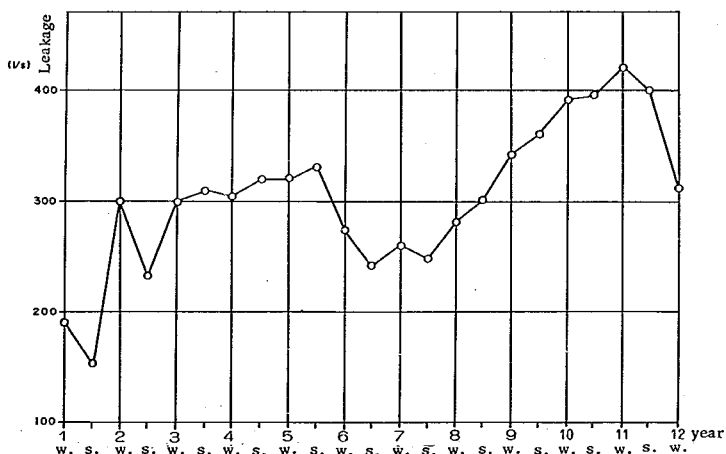


Fig. 19. Leakage in summer and winter in H Dam.
(All leakage values for water level 61.5m)

is 61.5 m (202') in both summer and winter.) The leakage was rather large in winter and rather small in summer within two years after the completion of the dam, as is the case with concrete dams. Contraction joints in concrete dams open in winter because of the temperature shrinkage of concrete. The leakage increased slightly in a period between the 3rd year and 5.5th year after completion of the dam, decreased in a period between 5.5th year and 7.5th year, and increased again in a period between 7.5th year and 11th year. The cause of such behavior is not known yet.

Fig. 20 shows the relationship between water level of the reservoir and the leakage in 7th, 9th and 10th year after the completion of the dam. There is a linear relationship between the water level and the leakage in each year. This means that the leakage is due to the concrete facing slab or shallow depths of the foundation. Fig. 20 shows also the leakage increases year by year. The leakage through the concrete slab does not decrease which is in contrast to the leakage in foundation or impervious zone of center core embankment dams which may have self-curing properties. The reservoir of H Dam was emptied in the 12th year after completion of

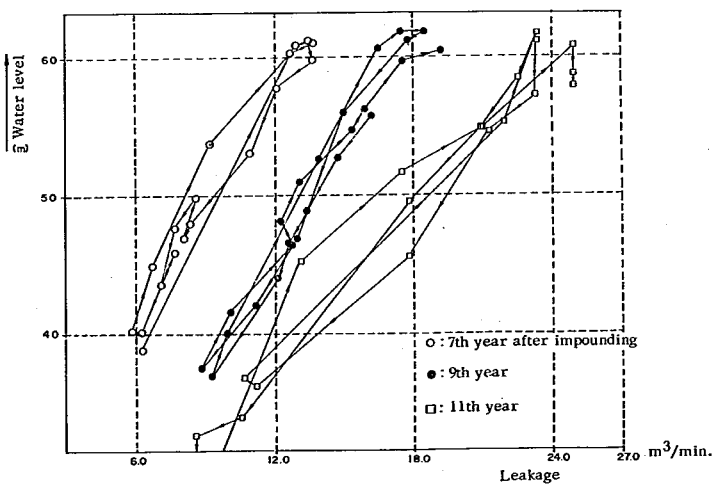


Fig. 20. Relationship between leakage and water level in H Dam.

the dam so that the concrete facing slab might be repaired.

1-5. Summary of Measurements of the Leakage

The instrumentations and procedures to measure the leakage in rockfill dams have been discussed above. The followings are the conclusions of this chapter from the point of view of the safety control of the rockfill dams.

1. The downstream weir system as shown in Fig. 1 is easily affected by the surface water. The surface water which flows in the weir should be suppressed as much as possible. The discharge through the weir should be observed at least for a year before the initiation of the first filling of the reservoir in order to understand the effects of precipitation.
2. Instrumentations and procedures as shown in Fig. 6 and Fig. 9 are the recommended system to measure the leakage through rockfill dam bodies. This system can measure the leakage of the dams without being affected by the surface water or precipitation.
3. The seepage flow through shallow depths of the foundation is more important than in the dam body. Instrumentation as shown in Fig. 11 is effective in measuring the seepage flow through depths of the foundations. The valve of the observation hole should be closed usually and the water pressure in the observation hole is read at the pressure gauge. The leakage through the observation hole should be measured every day during the first filling of the reservoir. If the discharge of a observation hole exceeds a certain limit (about 100 l/min (26 gallon/min)), the rate of the filling of the reservoir should be controlled and the cause of the leakage should be investigated primarily considering the geological condition of the foundation. Regrouting should be carried out if the leakage is very large. We will be able to insure the safety of the foundation, against seepage if we take these procedures.
4. The leakage through the dam body and foundation is apt to decrease year by year. Therefore it is very important to observe the leakage during the first filling of the reservoir.
5. The leakage in rockfill dams with facing slab may increase with time in

contrast with the properties stated in 4. Therefore such consideration should be made at the planning stage that it is necessary to empty the reservoir in order to repair the facing slab at long intervals.

2. Deformation of Rockfill Dams

The deformation measurement of rockfill dams may be classified into two types according to the position of the survey point. One is surface deformation and the other is internal deformation. Surface deformation is obtained by surveying the measurement points as shown in Fig. 21. The internal deformation will be obtained using the instruments as shown in Fig. 22 or other instruments such as inclinometers.

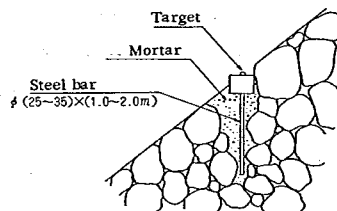


Fig. 21. Target to measure surface deformation.

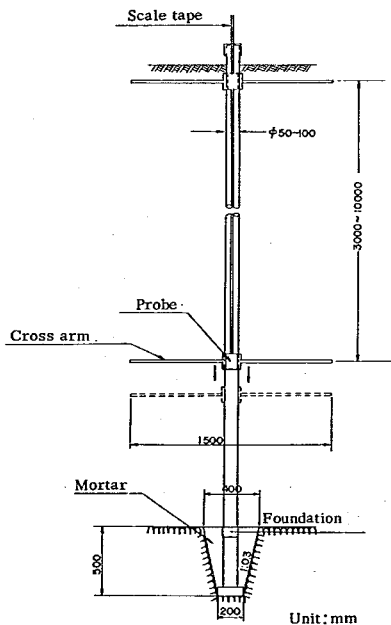


Fig. 22. Installation of internal vertical movement device

2-1. Surface Deformation

Five to ten located measurement points are usually along the crest and five to fifteen measurement points are located on the downstream slope of a rockfill dam.

Fig. 23 shows the settlement and the horizontal movement of the center of the crest and the water level of the reservoir of A Dam. The settlement was about 10 cm (4 in.), when the water level of the reservoir reached the normal water level six months after the initiation of impounding. The rate of the settlement decreased a little in subsequent six months when the water level was maintained nearly constant. The settlement accelerated again after 1 year and 2 months, when the water level was lowered. The crest of the dam settled about 7.5 cm (3 in.) in this period.

The horizontal movement is proportional to the water pressure. The crest of the dam moves downstream when the water level goes up and moves upstream when the water level goes down.

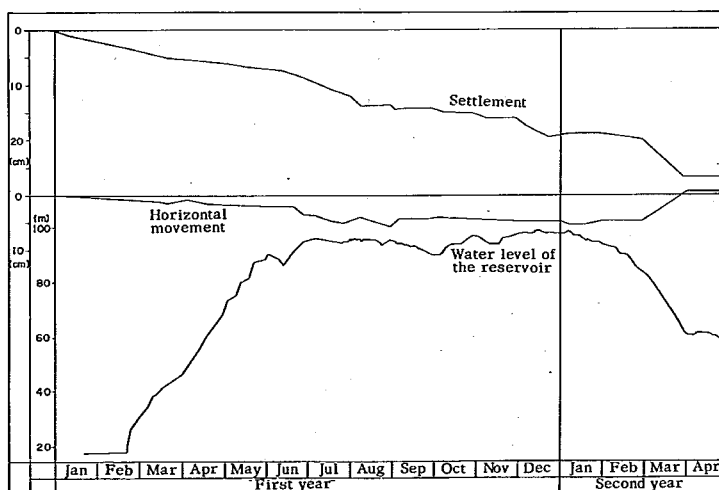


Fig. 23. Deformation at the center of the crest in A Dam.

Fig. 24 shows the settlement and horizontal movement of the center of the crest and the water level of D Dam (see Fig. 1 for dam cross section). The

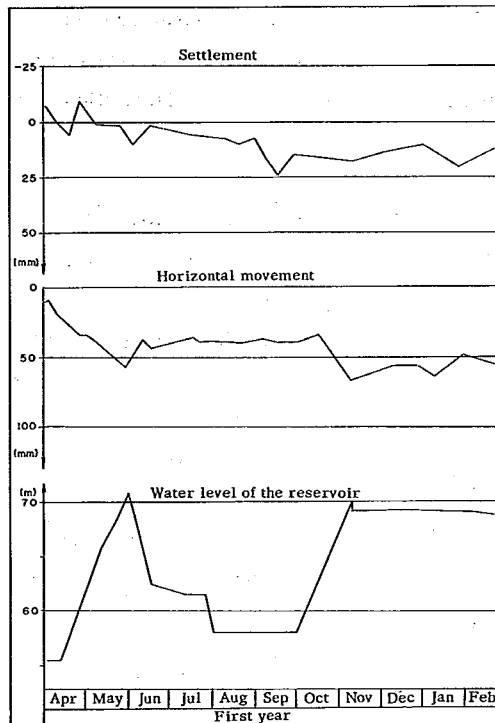


Fig. 24. Deformation at the center of the crest in D Dam.

settlement is very small and is about 2.5 cm (1 in.) at maximum. The rate of the settlement is large when the water level goes down. This is the same tendency as in Fig. 23. D Dam had been completed one year before the filling of the reservoir started. The year of consolidation partially explains why the settlement is so small. The horizontal movement is proportional to the water level as in Fig. 23.

Figs. 23 and 24 shows that the settlement of the rockfill dam has the property of consolidation and the dam body is subjected to the upward vertical force which is due to the buoyancy, when the water level goes up. The settlement is accelerated as a result of the increase of effective stresses in the dambody when the water level goes dam. The horizontal deformation consists of elastic deformation and plastic

deformation. The former is small in well compacted embankment dams.

The deformation described above is only concerned with the deformation at one measurement point of the crest. Next the distribution of deformation along the crest will be shown.

Fig. 25 shows the settlement and horizontal deformation along the crest of E Dam. The maximum of horizontal movement is about 10 cm (4 in.) when the reservoir is filled full. The horizontal movement does not increase after that, while the settlement increases as time goes on.

Fig. 26 shows the settlement and the horizontal movement of B Dam during 13 years. The settlement at the center of the crest is 35 cm (13.8 in.) after one year and 65 cm (25.6 in.) after 14 years. The horizontal movement is 41 cm (16.1 in.) after one year and 54 cm (21.2 in.) after 14 years.

Fig. 27 shows the settlement and horizontal deformation of a point on Miboro Dam. Figs. 26 also shows that the settlement has the property of creep and the horizontal movement consists of elastic deformation and plastic deformation as described before.

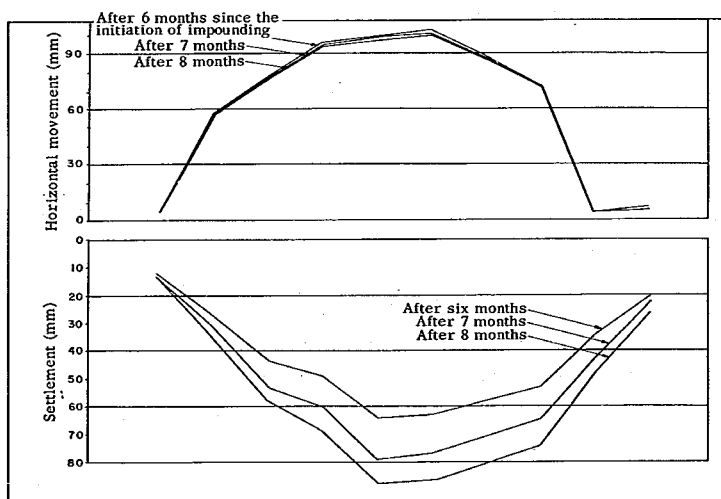


Fig. 25. Deformation along the crest in E Dam

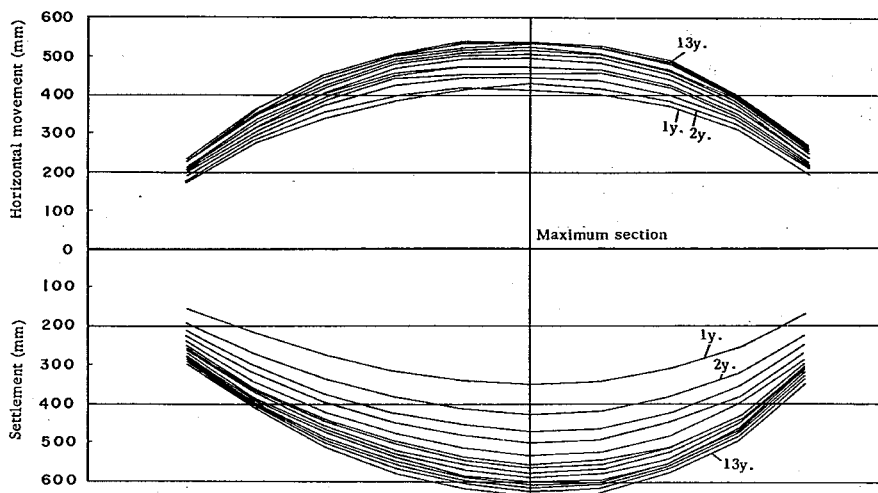


Fig. 26. Deformation along the crest in 14 years in C Dam.

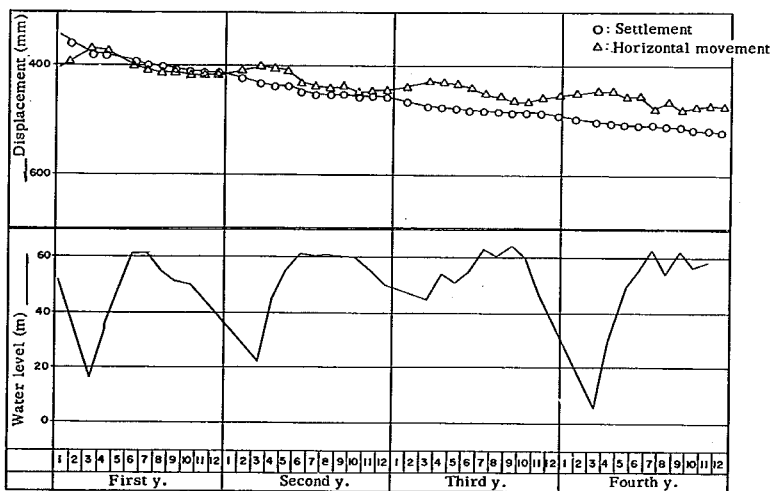


Fig. 27. Deformation at the center of the crest in C Dam.

Fig. 28 shows the deformation along the crest of B Dam during 7 years. The maximum settlement and horizontal deformation at the crest are 12 cm (4.7") and 8 cm (3.1") respectively after one year and 42 cm (16.5") and 13 cm (5.1") respectively after 7 years. It should be noticed that the maximum settlement does not occur at the maximum section but closer to the right abutment.

The deformation of C Dam as shown in Figs. 26 and 27 is larger than those of A, D and E Dam as shown in Figs. 22 to 24. The reason for this might be attributed to the construction method. The rockfill zones were compacted in thin layers in A, D and E Dam, while they were not in C Dam.

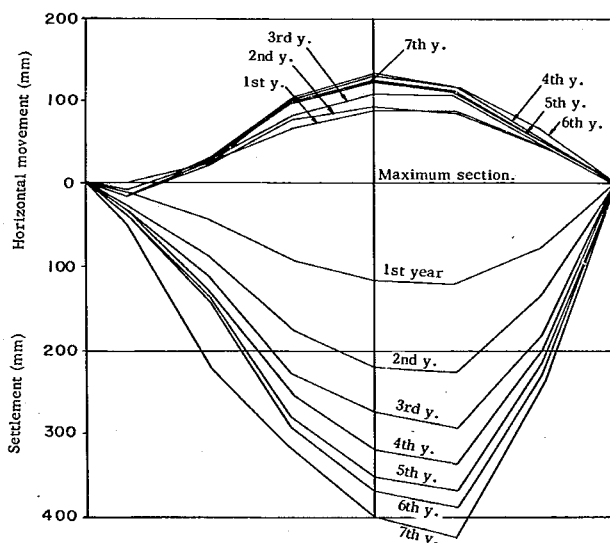


Fig. 28. Deformation along the crest in B Dam.

Finally the ratios of the maximum deformations at the crest to the height of the dam are shown in Table 1.

Table 1. Deformation (percent of dam height)

Dam	$\frac{\text{Settlement}^{1)} \times 100\%}{\text{Height of Dam}}$	$\frac{\text{Horizontal Movement}^{2)} \times 100\%}{\text{Height of Dam}}$
A	0.2	0.05
B	0.1	0.08
C	0.3	0.32
D	0.2	0.09
E	0.1	0.06

- 1) When 1 year passed since placement had completed.
 2) When the reservoir was first filled full.

2-2. Internal Deformation

Fig. 29 shows the arrangement of the internal vertical movement devices in D Dam. Fig. 30 shows the measured results of internal movement device No. 9 as shown in Fig. 29. The abscissa indicates the time after the initiation of placement, and the ordinate means the compressive deformation between two cross arms. For example, notation 1-2 means the compressive deformation between crossarm 1 and 2. In Fig. 30, the compressive deformation fluctuates within 2.5 cm (1"), after the filling of the reservoir was initiated. This phenomenon may be caused by the buoyancy of the water pressure and corresponds to the surface deformation as shown in Fig. 24.

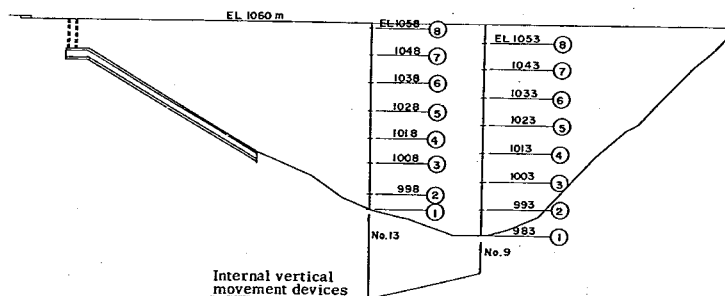


Fig. 29. Arrangement of the internal vertical movement devices in D Dam.

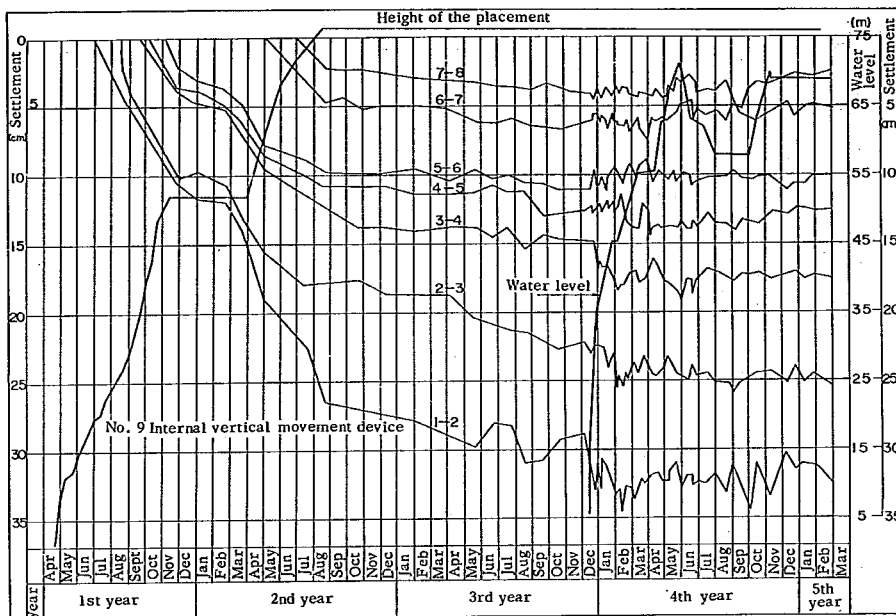


Fig. 30. Internal vertical movements in D Dam.

Fig. 31 shows the relationship between the compressive and the overburden pressure which is derived from Fig. 30.

Next relative settlements among different zones in rockfill dams will be shown. If the materials and the compaction methods are different among zones, differential settlements will occur and the boundary between the zones may be sheared.

Fig. 32 shows the relative settlements which were measured using the inclinometers (E Dam). Absolute settlements cannot be known, because the angles are measured. The differential settlement at the end of the impervious zone is shown in Fig. 32. The placement of the impervious zone in E Dam is like this: thickness of one placement: 20 cm (7.9"), compacted 12 times by sheep's foot roller, while the placement of the filter zone is: thickness of one placement: 30 cm (11.8"), compacted 4 times by bulldozer. Since there were gaps at the boundary between the impervious and filter zones, the upstream and downstream ends of the impervious

zone may not have compacted sufficiently.

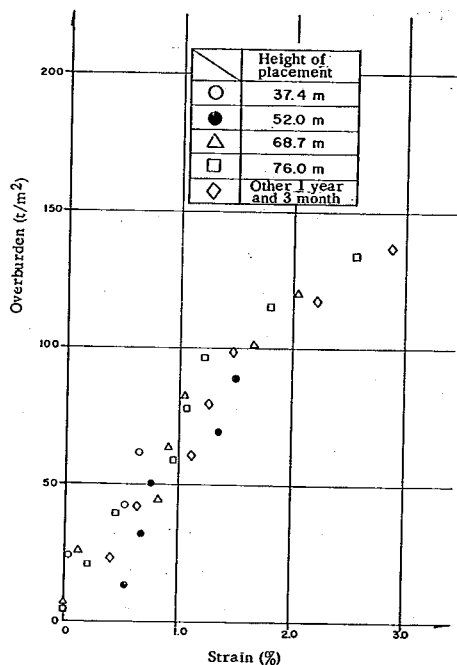


Fig. 31. Relationship between overburden load and strain.

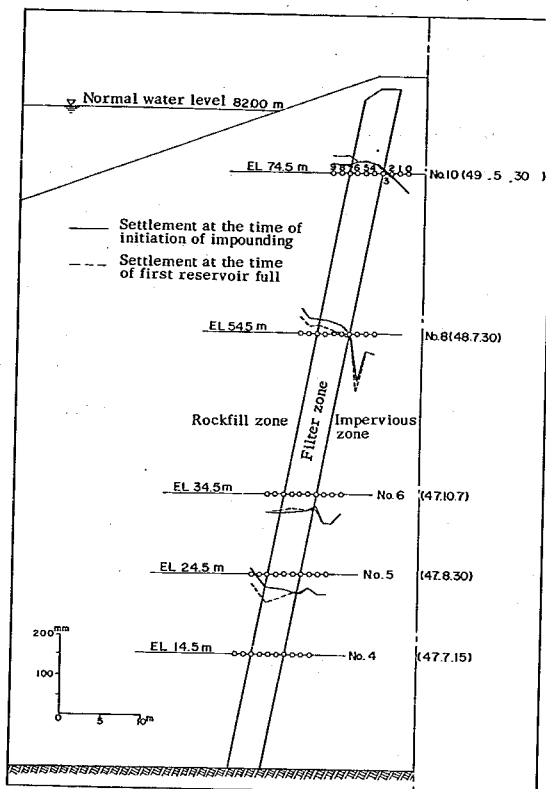


Fig. 32. Horizontal distribution of settlement in E Dam.

2-3. Summary of Measurements of Deformation

As the stress strain properties of concrete dams (concrete) are more elastic than that of rockfill dams, the deformations have more definite meaning in concrete dams than in rockfill dams. The deformations of concrete dams can be used as indices for the safety control of dams. In rockfill dams, however, the characteristics of materials are complicated and affected by the property of the material and the method of construction. It would be difficult to define the limits of the deforma-

tions in rockfill dams. Therefore it is important to pay attention to the displacement distribution and the longterm changes of displacement for the safety control of rockfill dams.

Nobori and Duncan (1972) reviewed published case histories of movements in embankment dams caused by reservoir filling. They distinguished movements in embankment dams during reservoir filling into three different types:

- (1) compression due to wetting in the foundation causes settlement; because the upstream portion of the foundation is wetted first, this settlement is not uniform;
- (2) compression due to wetting of homogeneous dams or the upstream shell of zoned dams causes settlement of the upstream portion and upstream movement of dam, with a general spreading of the dam and possible longitudinal cracks; and (3) the water load causes downstream movements; because the water load increases proportionally to the square of the depth of water retained, downstream movements are most pronounced during the latter stages of reservoir filling.

The observed field results reported here do not show the two types of movements (1) and (2) stated above. The reason why they do not show may be attributed to three factors; (1) the foundations consist of hard and sound rock; (2) impervious zones consist of well graded materials which include coarse sand and gravel and are well compacted at a slightly higher water content than optimum water content; (3) pervious zones consist of well graded hard rock materials and are well compacted. The compression due to wetting in embankment dams may not be a major factor of movements in case the foundations and embankments meet the conditions stated above.

The characteristics of deformations in rockfill dams investigated in this report are as follows:

1. Settlement has the property of creep. The crest of the rockfill dam settles usually 0.1 to 0.3 % of the dam height one year after the completion of placement. The settlement goes on subsequently. The crest will settle 2 to 4 times more than that of the first year after ten years.
2. The settlement is not so sensitive to the water level as is the horizontal deformation. The rate of settlement decreases when the water level rises increasing buoyancy effects, and increases when the water level

drops increasing effective stresses.

3. The horizontal deformations of rockfill dams are proportional to the water level of the reservoir. The horizontal displacement at the center of the crest is about 0.05 to 0.3 % of the dam height when the reservoir is first filled full. Most of the horizontal deformation in well compacted rockfill dams is elastic. The crest of the dam moves in the upstream direction, when the water level goes down. The horizontal deformation does not proceed as much with time as the settlement. That is, the horizontal deformation consists of elastic deformations and plastic deformations.
4. The amount of deformation of rockfill dams varies much depending on materials and construction method. Deformation is less in rockfill dams which are constructed by thin layer rolled compaction method.
5. Measurements of surface deformation would be adequate for the safety control of rockfill dams. Measurement of internal deformation will give us information for the design.

3. Measurements of Pore Water Pressures

The pore water pressure in rock foundations has already been described in 1-3. Here, the measured results of pore water pressure in rockfill dam bodies will be shown.

Correlation was made between the observed results and the two dimensional seepage flow analysis.

3-1. Measurements of Pore Water Pressure

Fig. 33 shows the pore water pressure distribution of B Dam after four months since the initiation of impounding. The water pressure in upstream filter zone are

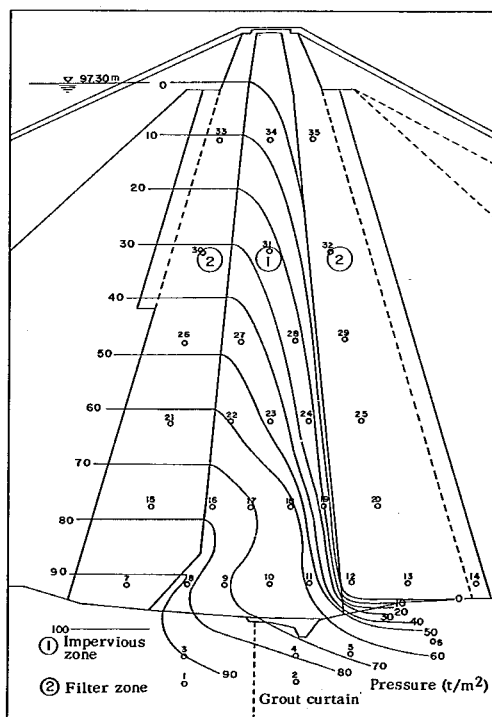


Fig. 33. Pore water pressure distribution in B Dam.

100 % of full reservoir pressure and those in downstream filter zone are 0 %. This shows that there is a large difference in permeability between impervious zone and filter zone. The hydraulic gradient is large in the downstream end of the core zone.

Fig. 34 shows the pore water distribution of A Dam after 1.5 years since the filling of the reservoir. Residual water pressures still remain at the bottom of the impervious zone. The impervious material of this dam has a high uniformity coefficient and a very small coefficient of permeability. Residual water pressures such as shown in Fig. 35 are not harmful to thin center core dams because there are no residual water pressures in filter zones and the impervious zone is subjected to high effective lateral earth pressures.

Fig. 35 shows the relationship between the water level of the reservoir and

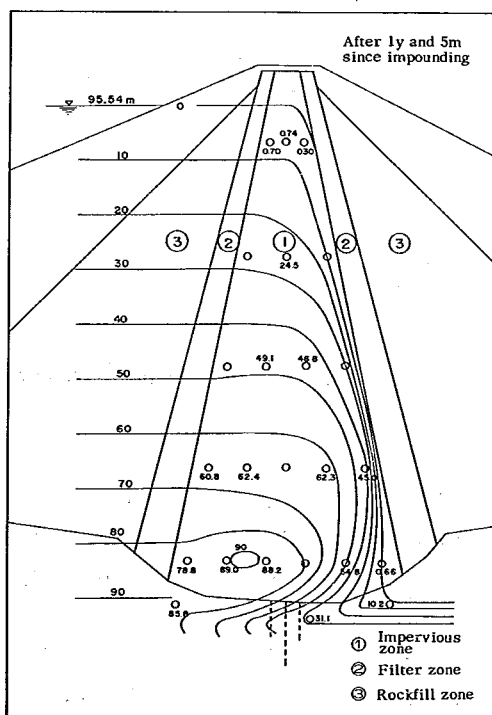


Fig. 34. Pore water pressure distribution in A Dam.

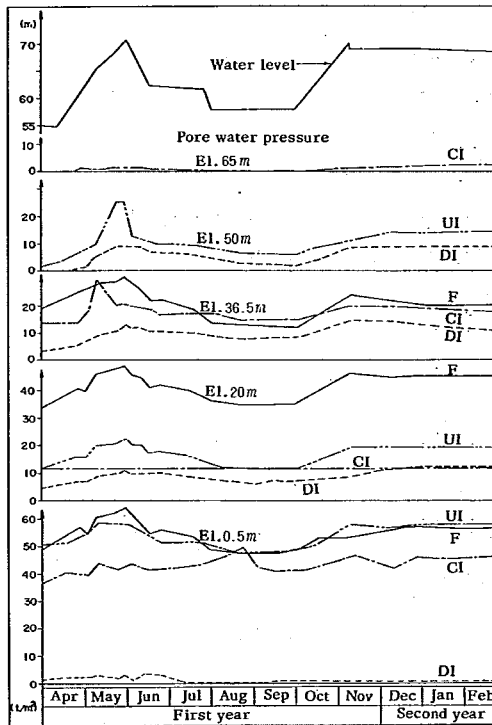


Fig. 35. Pore water pressure and water level of the reservoir in D Dam.

the pore water pressures in D Dam. Each pore water pressure corresponds well to the water level.

3-2. Analysis of Two Dimensional Steady State Seepage Flow

The seepage flow in rockfill dam bodies can be assumed to be applicable for Darcy's rule. The two dimensional steady state seepage flow analysis was performed using the finite element method.

Fig. 36 shows the observed field results of E Dam. Fig. 37 shows the finite element model of the impervious zone and foundation of E Dam.

Fig. 38 shows the water pressure distribution determined by the analysis, assuming the coefficient of the permeability of impervious zone, grout curtain and

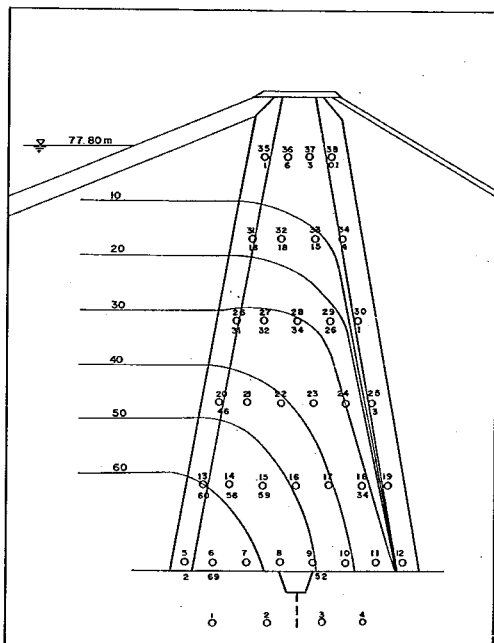


Fig. 36. Pore water pressure distribution in E Dam.

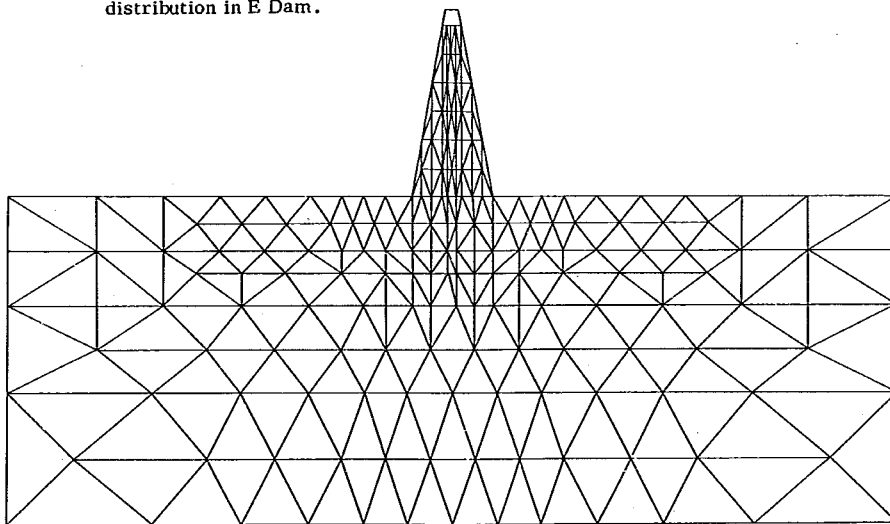


Fig. 37. Finite element discretization of the model.

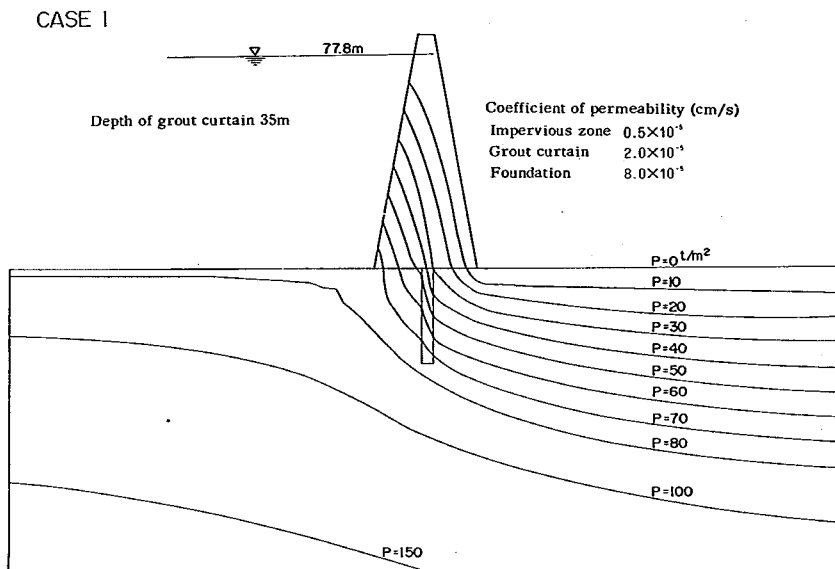


Fig. 38. Water pressure distribution in Case 1.

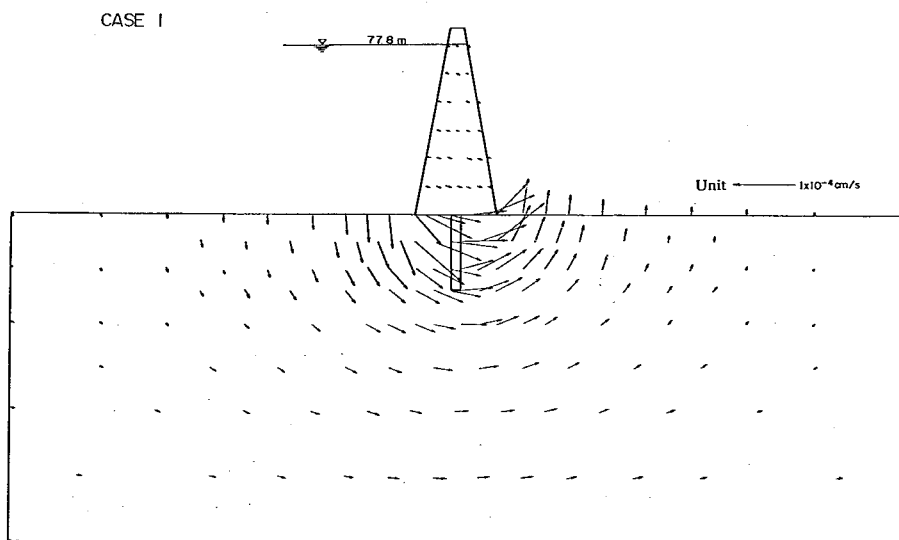


Fig. 39. Velocity distribution in Case 1.

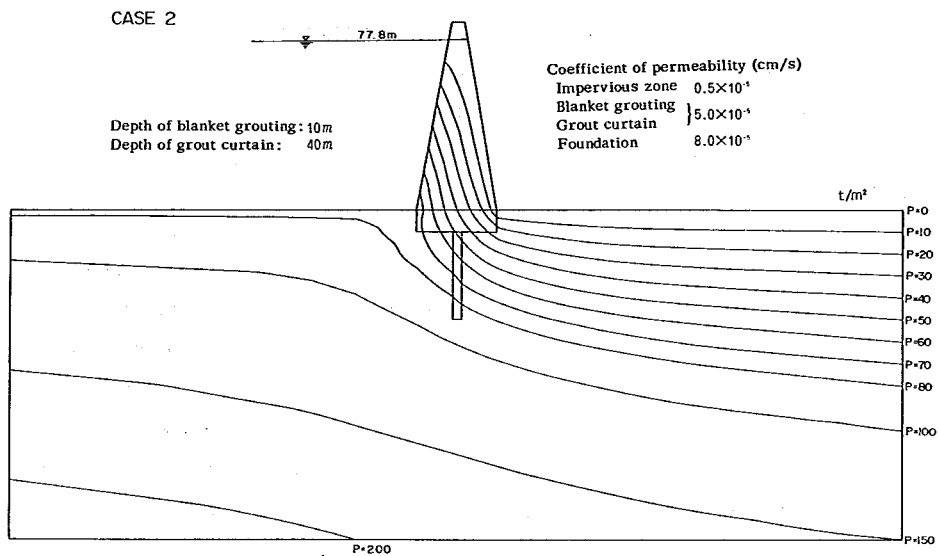


Fig. 40. Water pressure distribution in Case 2.

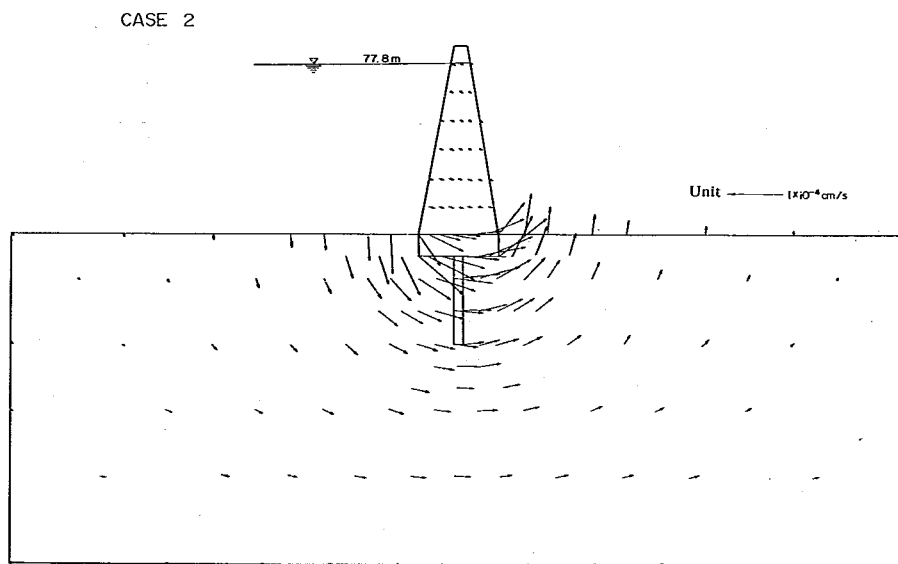


Fig. 41. Velocity distribution of Case 2.

foundations to be 0.5×10^{-5} cm/s, 2.0×10^{-5} cm/s and 8.0×10^{-5} cm/s respectively (case 1). Fig. 39 shows the velocity distribution in case 1. The analysis shows that the amount of seepage flow is larger in the foundation than in the dam body and the water pressure does not drop so much behind grout curtain under assumed conditions.

Fig. 40 and 41 show the results of case 2 which has blanket grouting. Blanket grouting is usually performed prior to the curtain grouting in the foundation which contacts the impervious zone. The effects of grouting are not so clear in case 2, because the coefficient of permeability of grout curtain and blanket grouting are rather large. Table 2 shows the discharges of seepage flow per one meter length of the dam in case 1 and case 2. If the length of the dam is 100 m (assuming the height to be uniform), the total discharge of leakage in the dam body and foundation are about 30 l/min (8 gallon/min), 200 l/min (53 gallon/min) respectively, which

Table 2. Discharge of seepage flow (by FEM Analysis)

CASE 1						CASE 2					
	Joint No.	Width	A (cm ²)	u (cm/s)	Q (cm ³ /min)		Joint No.	Width	A (cm ²)	u (cm/s)	Q (cm ³ /min)
Dam body	35	12.9	12.9×10^4	4.69×10^{-6}	36.3	Dam body	24	12.9	12.9×10^4	4.76×10^{-6}	36.8
	42	25.9	25.9×10^4	8.21×10^{-6}	127.6		31	25.9	25.9×10^4	8.22×10^{-6}	127.7
	47	26.0	26.0×10^4	1.01×10^{-5}	157.6		36	26.0	26.0×10^4	1.03×10^{-5}	160.7
Foundation	50	5.9	5.9×10^4	7.59×10^{-5}	268.7	Foundation	39	5.9	5.9×10^4	7.19×10^{-5}	254.5
	116	10.9	10.9×10^4	6.63×10^{-5}	433.6		66	10.9	10.9×10^4	7.35×10^{-5}	480.7
	117	10.0	10.0×10^4	4.83×10^{-5}	289.8		67	15.0	15.0×10^4	4.98×10^{-5}	448.2
	118	12.5	12.5×10^4	3.74×10^{-5}	280.5		68	17.5	17.5×10^4	3.26×10^{-5}	342.3
	119	17.5	17.5×10^4	2.79×10^{-5}	292.3		69	25.0	25.0×10^4	2.35×10^{-5}	352.5
	120	20.0	20.0×10^4	1.94×10^{-5}	232.8		70	37.5	37.5×10^4	1.33×10^{-5}	299.3
	21	22.5	22.5×10^4	1.39×10^{-5}	187.7						
Leakage through dam body 0.285 l/min						Leakage through dam body 0.325 l/min					
Leakage through foundation 1.985 l/min						Leakage through foundation 2.178 l/min					

coincide approximately with the observed results of the dam as shown in Fig. 36 (ref. Fig. 8).

3-3. Summary of Measurements of Pore Water Pressure

1. The residual pore water pressure is so small in the rockfill dams shown above that it does not affect the stability of the dambody, if the impervious materials are well graded and the impervious zone is thin. If the material of the rockfill is fine and silty, the measurements of pore water pressure during construction are very important.
2. Pore water pressure devices are usually located in one or two typical sections of rockfill dams. The observed results of pore pressure, however, show the only local behavior of the dam, while the deformation at the center of the crest of the dam and the leakage show the integrated behavior of the dam. The pore water pressure devices can easily malfunction and we cannot judge the safety of the dam if one pore water pressure device shows an extraordinary value. Accordingly measurements of pore water pressures in dam bodies are not so useful for the safety control of the rockfill dams.

Materials in rockfill dams will be weathered for many years and might lose their strength, if the materials are not sound. Weathering of rockfill materials may be influenced by many factors: including moisture change (rising and falling of the water level), temperature change, stress condition and etc. Here temperature change was picked up. Temperature change was measured in E Dam in order to know the location and magnitude of temperature change inside the dam.

- 38 -

It should be noted that the site of this dam is in a northern part of Japan and on a high altitude where the temperature sometimes goes down below -30°C (-22°F) in the winter. The farther inside the dam the measurement point is, the higher the temperature is, when the filling of the reservoir started in November. The temperature in upstream side is higher than in downstream side, because the upstream slope looks towards south and gets more sunlight. When the water level goes up to the elevation where the thermometers were set up (after six months since starting the filling of the reservoir), every thermometer in upstream side shows the same temperature which is equal to the temperature of the water of the reservoir. In downstream side, the closer to the dam surface the thermometer was, the higher the summer temperature was because the outer part of the dam body is warmed by the open air. Accordingly the downstream rockfill zone will continue seasonal change of temperature, and the temperature change is a maximum at the outer area. Since there is a time lag of the temperature change between the inner area and outer area, there is a temperature gradient in dam body. Fig. 44 shows the maximum temperature change in one year. A schematic view of temperature change in the dam body will be shown in Fig. 45. In Fig. 45, zone 1 and 2 are subject to

Upstream Rockfill		Downstream Rockfill	
Measurement Point	Temperature Difference	Measurement Point	Temperature Difference
No. 71	5.7°	No. 91	—
72	15.4	92	23.0°
73	15.0	93	14.7
74	12.1	94	—
75	11.6	95	9.4
76	10.4	96	6.1
77	7.8	97	—

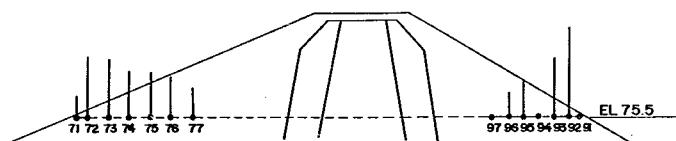


Fig. 44. Maximum temperature change in a year.

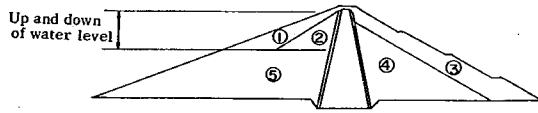


Fig. 45. Temperature change distribution in dambody.

big temperature change affected by the rise and fall of the water level. The temperature change in zone 3 is rather large because it is affected by the temperature change of open air. The temperature in zone 5 is always equal to that of the water of the reservoir, and its temperature change will be small.

These results should be taken into account in zoning of rockfill dams.

5. Measurements for the Safety Control of Dams

Table-3 shows the obligatory items of measurements of dams in Japan as provided by a government ordinance. Extra measurements besides Table-3 are required for dams which height are over 100 m (328') or have special structures, if necessary.

Table 3. Measurement items according to type of dams

Type of Dams	Measurement Items
Concrete Gravity Dam, over 50 m	Leakage, Uplift
under 50 m	leakage, Deformation, Uplift
Concrete Arch Dam, over 30 m	Leakage, Deformation
under 30 m	Leakage, Deformation, Uplift
Embankment Dam, homogenous type	Leakage, Deformation, Seepage Line
zoned type	Leakage, Deformation

The Japanese National Committee on Large Dams requires the frequencies of measurement in dams as shown in Table-4. The intervals of the measurements are close in the first period, when we have to know whether the dam shows the stable behavior as expected during the first filling of the reservoir. In the third period, the deteriorations such as the deterioration of concrete and spillways and outlet works come into the problem.

The measurements in the first period are the most important for the safety control of dams. The measurements of leakage should take first priority in embankment dams among various kinds of measurements as stated in previous chapters. It is recommended to have inspection galleries in the foundation of rock-fill dams for this purpose. If we have inspection galleries, we can know the places of leakage through foundation and repair it by regrouting.

Table 4. A interval of measurements

		Gravity D. 100m Arch D. 30m	Gravity D. ¹⁾ 100m Arch D. ²⁾ 30m	Embankment D.	
				Homogeneous Type	Zoned Type
Leakage	1st. period	a day	a day	a day	a day
	2nd. period	a week	a week	a week	a week
	3rd. period	a month	a month	a month	a month
Uplift	1st. period	a week	a week	None	None
	2nd. period	a month	a month		
	3rd. period	3 months	3 months		
Seepage line	1st. period	None	None	a week	None
	2nd. period			a month	
	3rd. period			3 months	
Deformation	1st. period	a day	a week	a week	a week
	2nd. period	a week	a month	a month	a month
	3rd. period	a month	3 months	3 months	3 months

1) Gravity D. which height is under 50m does not need to measure deformation.

2) Arch D. which height is under 30m does not need to measure uplift.

6. Acknowledgements

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