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3

CONSOLIDATION EFFECTS ON FOUNDATION PERMEABILITY

August, 1991

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CONSOLIDATION EFFECTS ON FOUNDATION PERMEABILITY

by

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Abstract :

The fill placement works on the dam foundation as overburden loading. This loading would influence the stress states in the foundation resulting in the change of strain and permeability in the underground. Therefore, the authors actually measured the deformation of the dam foundation and the change of permeability due to the fill placement at two damsites. The measured results for both damsites indicate that the fill placement causes the compression of dam foundations accompanying the decrease of permeability. The authors feel that these results are very helpful for the determination of effective grouting design for the foundation of fill dams. Furthermore, the authors theoretically try to explain the relationship between the compressive strain of the foundations and the decrease of permeability due to the fill placement by using simple models.

Keywords : dam foundation, seepage control, permeability, deformability.

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1. INTRODUCTION

Controlling seepage in the dam foundation is one of the most important and challenging parts of dam engineering. There are three basic methods for controlling seepage, i.e. reduction of seepage, drainage and the use of filters. As for rock foundations grouting is the most frequently used of the several seepage reduction methods possible. Foundation grouting at dams takes the form of curtain and blanket groutings. The curtain grouting is provided down to a comparatively deep depth at the foundation for the purpose of cutting off the underseepage. Blanket grouting is provided at a comparatively shallow depth to make the area more impermeable or to consolidate fractured or jointed rocks.

Recently, in Japan, most of the sites which have satisfactory geological conditions for the construction of dams have been already developed and so dams tend to be constructed on sites that are not always satisfactory in terms of geological conditions. In selecting the type of dams, fill dams are often chosen for sites which have a low bearing capacity for concrete dams. Therefore, many damsites for fill dams under construction or in planning are composed of weak rocks such as soft, or weathered, or fractured rocks. Since higher grouting pressures cause fracturing or lifting of the surface of foundations, the allowable pressures should be kept low in weak rocks, and this sometimes create serious difficulties particularly in improving the permeability for the shallow depth of the foundation.

On the other hand, since weak rock foundations show large deformability, these foundations, especially in the shallow areas, are expected to deform due to loading of the fill placement. If the foundations are consolidated, the permeability of the foundations will decrease. The authors feel that investigations on the quantitative relationship between the compressive strain and the decrease of permeability are very useful for controlling seepage in dam foundations.

From the above-mentioned viewpoint, the authors try to reveal the relationship between the compressive strain and the decrease of permeability due to fill placement in fill dams through the in-situ measurement and theoretical studies. Although many researches investigating permeability and the grouting of dam foundations have been made, there has been little systematic research from the above-mentioned viewpoint(3). Furthermore, the installation of foundation galleries

at the base of fill dams has enabled in-situ data for consolidation effects on the permeability due to fill placement to be obtained.

2. MEASUREMENTS AT A DAM

(1) Geology of Damsite

A Dam is a 74m high rockfill dam with a central core, and that was constructed in 1983 by the Miyagi Prefectural Government. The foundation of this damsite mainly consists of soft rocks, such as sandstone, tuffaceous sandstone, tuff, tuff breccia and mudstone of the Miocene, Neogene Tertiary (Fig. 1). The measurements of the deformation of the dam foundation and the change of permeability due to fill placement were conducted at the higher elevation of the right abutment. Laboratory tests were also conducted for sandy tuff in the area. Undisturbed samples were used for these tests. The results of these tests are summarized in Table 1. The results of in-situ 30cm diameter plate loading tests are shown in Table 2.

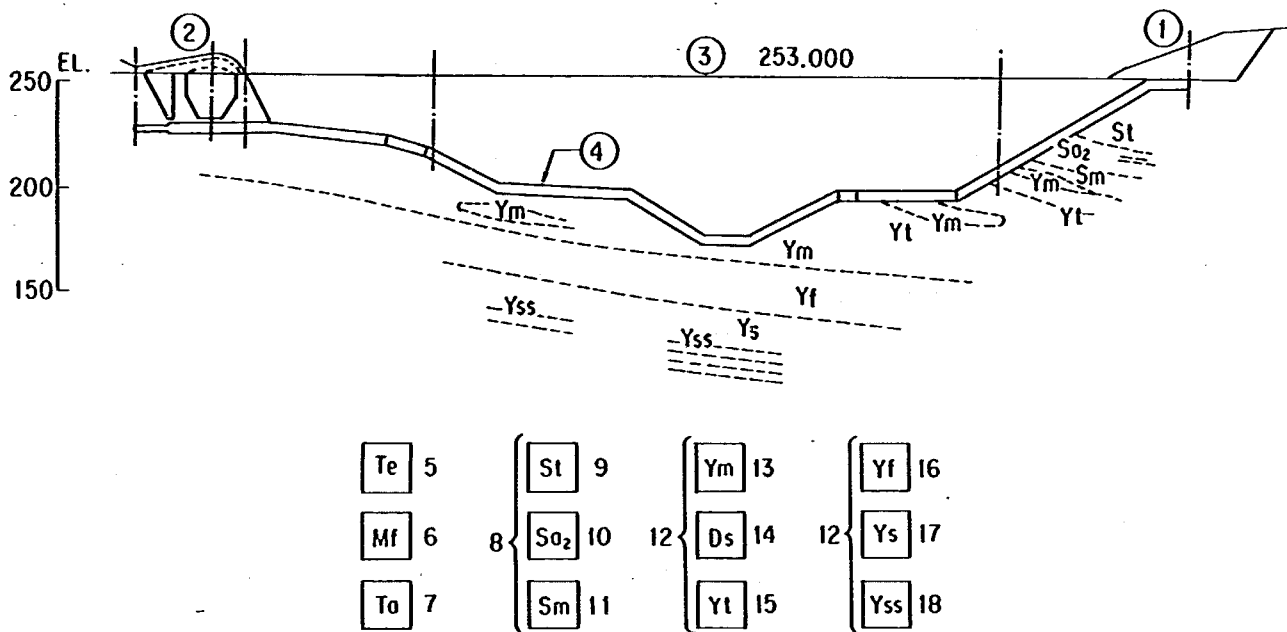


Fig. 1 Geological Map along Dam Axis (A Dam)

- | | |
|---|---|
| <ul style="list-style-type: none"> 1. Right Abutment 2. Left Abutment 3. Dam Crest 4. Gallery 5. Terrace Deposit 6. Mud-Flow Deposit 7. Talus Deposit 8. Shirasawa Strata 9. Sandy Tuff 10. Alternating Beds of Tuff and Mudstone | <ul style="list-style-type: none"> 11. Alternating Beds of Sandstone and Mudstone 12. Yumoto Strata 13. Alternating Beds of Sandstone and Mudstone 14. Dark Tuffaceous Sandstone 15. Tuff Breccia 16. Tuff 17. Tuffaceous Sandstone 18. Sandstone |
|---|---|

Table 1 Results of Laboratory Soil Tests (Sandy Tuff)

Wet density	1.62 t /m ³
Dry density	1.20 t /m ³
Natural water content	36.9%
Specific gravity	2.38
Void ratio	0.89
Unconfined compressive strength	7.7kgf/cm ² (755 k Pa)
Coefficient of permeability	2.7×10^{-5} cm/s

Table 2 Results of In-situ Plate Loading Tests (Sandy Tuff)

Coefficient of deformation	2,00kgf/cm ² (196MPa)
Tangential coefficient of elasticity	5,800kgf/cm ² (568MPa)
Secant coefficient of elasticity	5,100kgf/cm ² (500MPa)

(2) Instrumentation

Extensometers were installed and boreholes for water pressure tests drilled at the upper half elevation of the right abutment in order to measure the deformation of the foundation and the change of permeability due to fill placement. The location of extensometers is shown in Fig. 2. A pair of horizontal and vertical extensometers were installed every 6 to 7m in elevation. The initial length for measuring distance of an extensometer is 10m.

The location of boreholes for water pressure tests is illustrated in Fig. 3. These boreholes were drilled from the inspection gallery placed at the base of the dam. Pairs of horizontal and vertical boreholes were spaced at every 6 to 7m elevation consistent with the extensometers. Each borehole was 12m long, and water

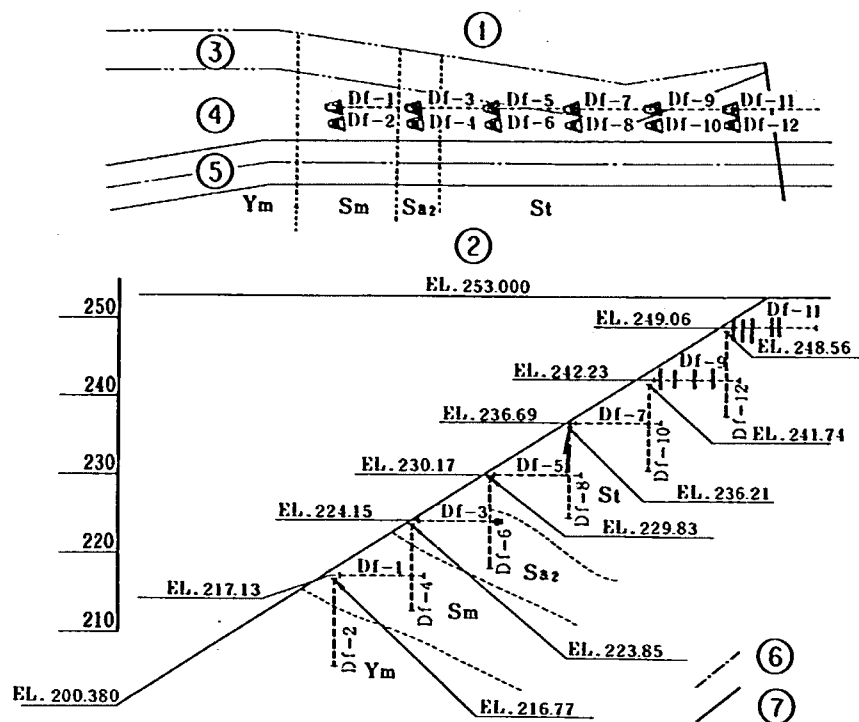


Fig. 2 Location of Extensometers (A Dam)

- | | |
|-------------------------|---------------------|
| 1. Plan | 5. Gallery |
| 2. Longitudinal Section | 6. Interbedded Clay |
| 3. Filter Zone | 7. Cement Grout |
| 4. Core Zone | |

pressure tests were performed at 3m long four stages in a borehole by using double packer method.

(3) Results of Measurements

a) the relationship between the height of fill placement and the displacement of foundation

The measured results of the deformation of the rock foundation are shown in Fig. 4. As the deformation of rock foundation was computed by using a two-dimensional model for the finite element method, the calculated values are also indicated in Fig. 4. The comparison of the calculated values and the measured results leads to the following :

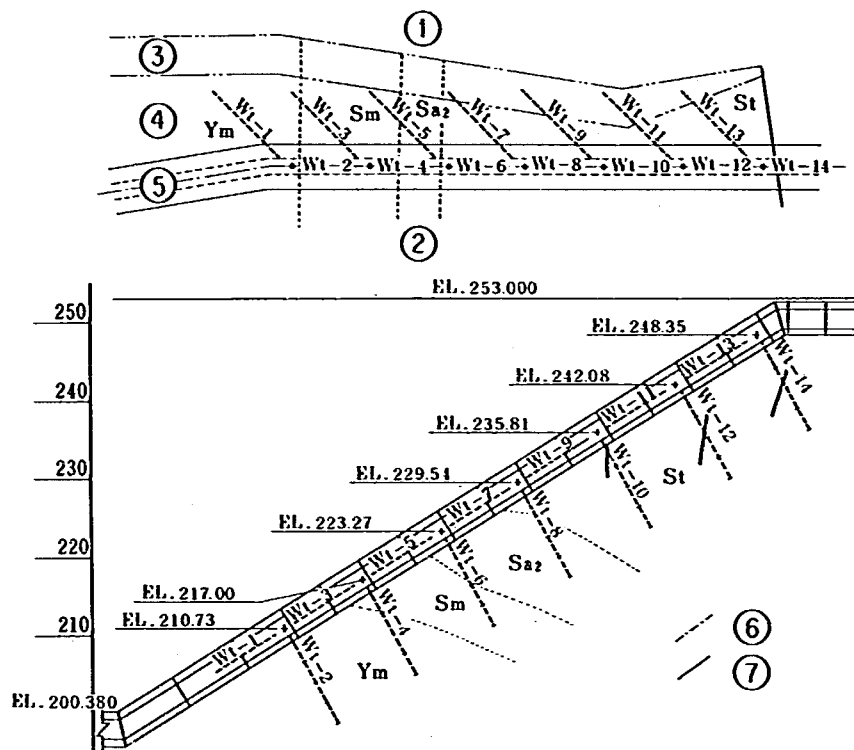


Fig. 3 Location of Boreholes for Water Pressure Tests (A Dam)

- | | |
|-------------------------|---------------------|
| 1. Plan | 5. Gallery |
| 2. Longitudinal Section | 6. Interbedded Clay |
| 3. Filter Zone | 7. Cement Grout |
| 4. Core Zone | |

1. As for the horizontal displacements, the calculated values coincided well with the measured results.

2. As for the vertical displacements, the calculated values are much larger than the measured results except for the results of Df-4 and 12. Larger predicted values are supposed to be derived from the limitations the of the two-dimensional model.

3. When the surface of fill placement reaches the vicinity of the elevation of each extensometer, the horizontal displacement turns to the tensile side. This tensile displacement indicates that the embankment loading functions temporarily as tensile force to the shallow portion of abutment near the surface of fill placement.

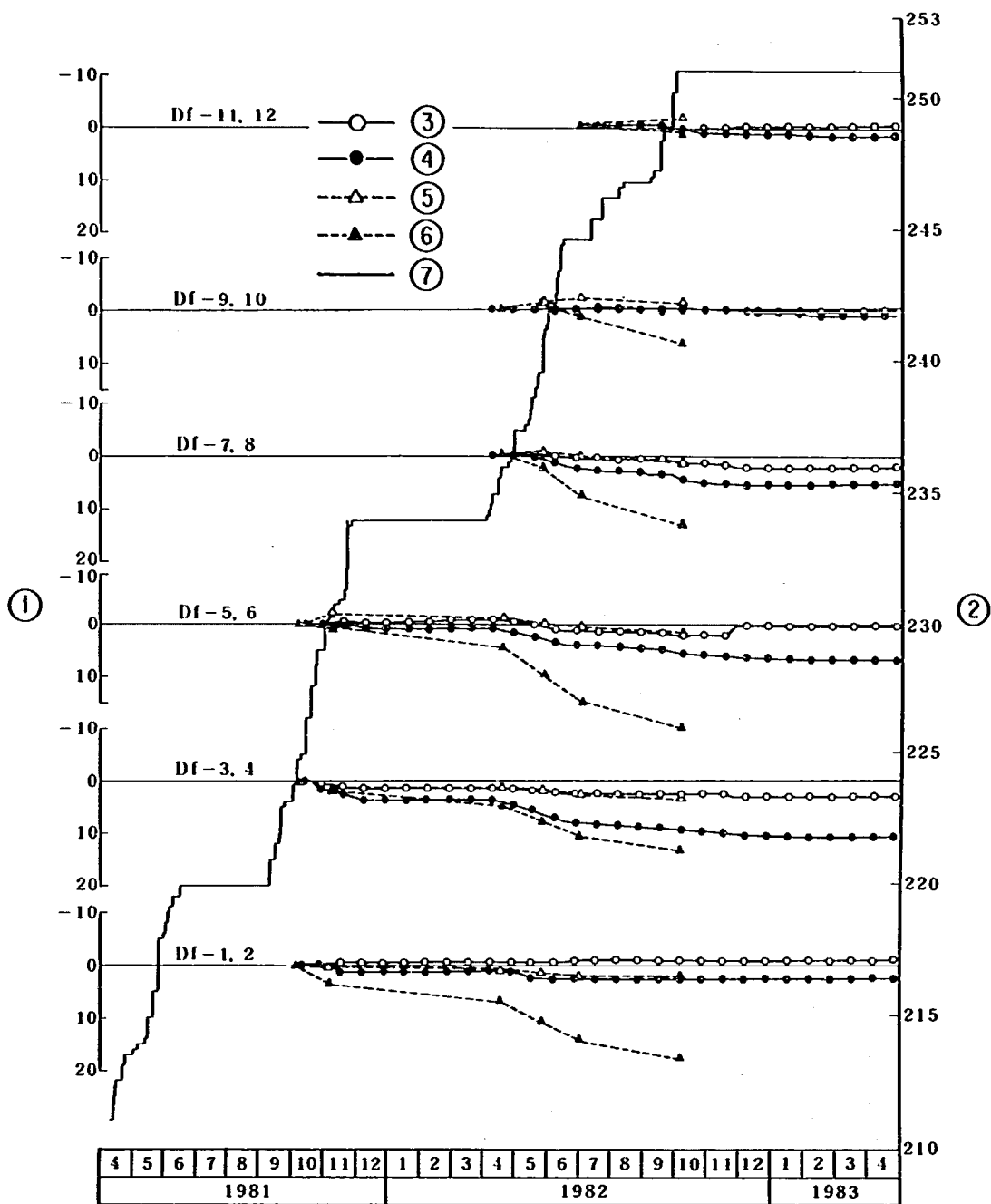


Fig. 4 Deformation of Foundation due to Fill Placement (A Dam)

- | | |
|-------------------------------------|---------------------------------------|
| 1. Displacement (mm) | 5. Calculated Horizontal Displacement |
| 2. Elevation (m) | 6. Calculated Vertical Displacement |
| 3. Observed Horizontal Displacement | 7. Height of Fill Placement |
| 4. Observed Vertical Displacement | |

Fig. 5 shows the relationship between the height of fill placement and the strain normal to the abutment surface at the location of the extensometer Df-7 and 8 computed from equation 1. In obtaining the strain in Fig. 5, the directions of principal stresses are assumed to be normal and in a direction parallel to the abutment surface.

$$\left. \begin{aligned} \epsilon_a &= \epsilon_x \cos^2 \theta + \epsilon_y \sin^2 \theta + \gamma_{xy} \sin \theta \cos \theta \\ \epsilon_\beta &= \epsilon_x \sin^2 \theta + \epsilon_y \cos^2 \theta - \gamma_{xy} \sin \theta \cos \theta \\ \gamma_{xy} &= (\epsilon_x - \epsilon_y) \tan 2\theta \end{aligned} \right\} \dots\dots\dots (1)$$

in which, ϵ_x =horizontal strain; ϵ_y =vertical strain; γ_{xy} =shear strain; θ = angle of inclination of the slope; ϵ_a = strain parallel to the slope and ϵ_β = strain normal to the slope. The compressive strains are assumed to be positive.

The elastic modulus of sandy tuff of the right abutment can be estimated to be approximately 3.3×10^3 kgf/cm² (330 MPa) from Fig. 5 by assuming a ratio of loading stress normal to the right abutment surface and the overburden weight to be 0.85(4). Here, 2.13tf/m³ (21 kN/m³) is used for the unit weight of core material in the calculation. The value of this elastic modulus mostly coincides with

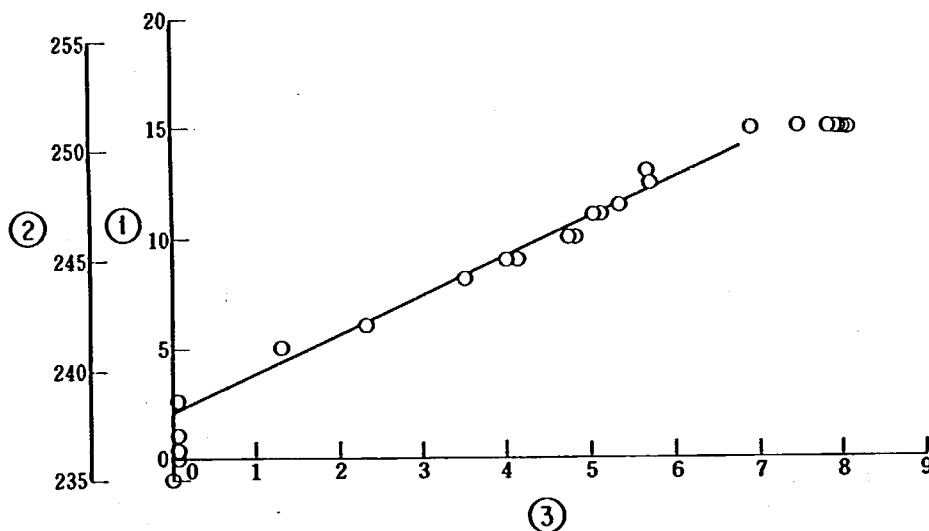


Fig. 5 Relationship between Strain Normal to Abutment and Height of Fill Placement (A Dam)

1. Height of Fill Placement (m)
2. Elevation (m)
3. Strain Normal to Abutment (X 10⁻⁴)

b) the relationship between the height of fill placement and the change of permeability

Fig. 6 shows the relationship between the height of fill placement and Lugeon values measured at the third stage of each borehole in the direction normal to the abutment surface. Though the boreholes for the water pressure tests were installed in several sorts of beds, such as Y_m , S_m , S_{22} and S_1 (Fig. 3), Lugeon values decrease with the increase of the height of fill placement. If water pressure tests are performed by using an injection pressure larger than the critical pressure, the excess pressure produces fracturing for the rocks surrounding the boreholes. Therefore, some of Lugeon values increased compared to the previous tests performed at the same stages of boreholes due to this hydraulic fracturing. However, Lugeon values tend to decrease with the increase of fill placement thereafter. This is because the overburden loading consolidated the foundation,

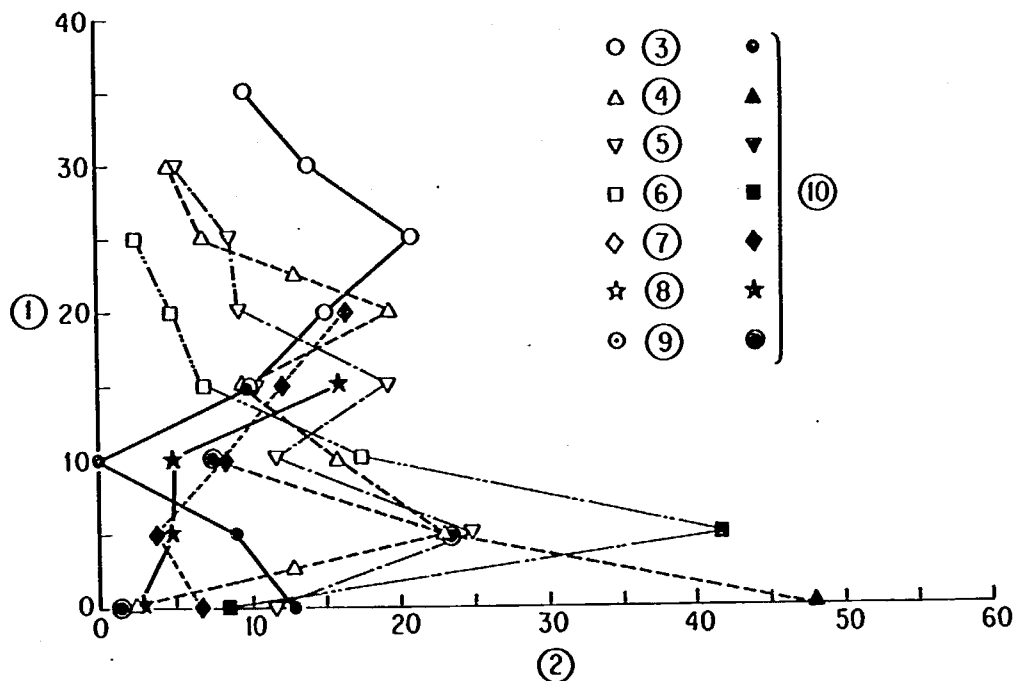


Fig. 6 Change of Permeability due to Fill Placement (Normal to Abutment, 3rd Stage, A Dam)

1. Height of Fill Placement (m)
2. Lugeon Value
3. Wt-2
4. Wt-4
5. Wt-6
6. Wt-8
7. Wt-10
8. Wt-12
9. Wt-14
10. Recognition of Critical Pressure

thus reducing the permeability. Test results for the boreholes in the direction parallel to the abutment surface also showed similar consolidation effects as in Fig. 6.

c) the relationship between the strain in the foundation and the permeability

The measured results in the previous section revealed that the rock foundation is consolidated due to fill placement, and that the permeability of the foundation decreases with the increase of fill placement. In this section, the measured data are rearranged with respect to the relationship between the strain in the foundation and the permeability. In the rearrangement of data, Lugeon values obtained from the tests observed critical pressure are excluded, for the rocks surrounding the boreholes are supposed to be fractured by that excess pressure. Fig. 7 illustrates the relationship between the Lugeon values obtained from the water pressure tests conducted in those boreholes drilled normal to the abutment surface, and the strain in the same direction. These strain values are obtained from the substituting horizontal and vertical strain values measured with the extensometers for equation 1. This figure indicates that the Lugeon values tend to decrease with the increase of compressive strain values. However, some of Lugeon values increase suddenly with an increase in the strain, which is possibly induced from the trouble of the packer in water pressure tests or the effects of horizontal strain.

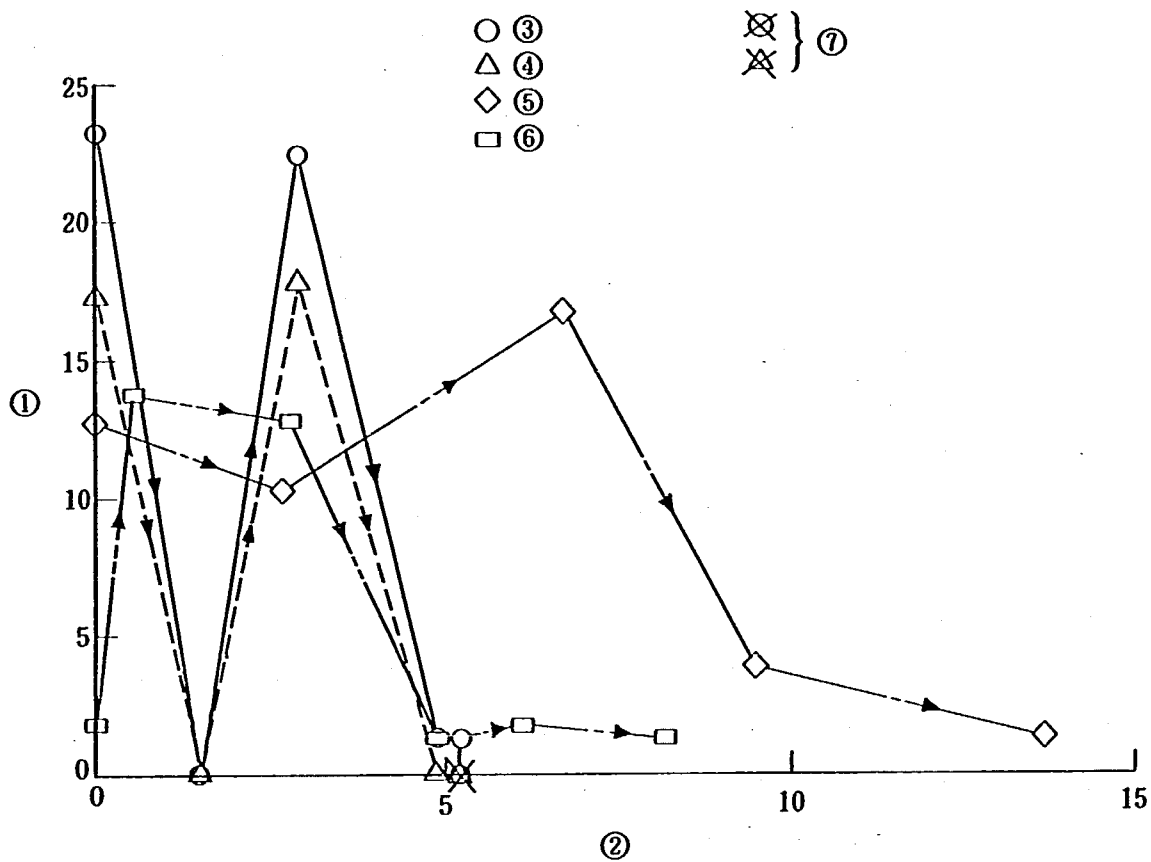


Fig. 7 Relationship between Lugeon Value and Strain Normal to Abutment (A Dam)

1. Lugeon Value
2. Strain Normal to Abutment ϵ_p ($\times 10^{-4}$)
3. Wt-4 1st stage
4. Wt-4 2nd stage
5. Wt-6 1st stage
6. Wt-8 1st stage
7. After Curtain Grouting Work

3. MEASUREMENTS AT B DAM

(1) Geology of Damsite

B Dam built in 1984 by the Fukushima Prefectural Government is a 96m high rockfill dam with a central earth core. The foundation of the damsite is composed of green schist and quartz schist of the Palaeozoic Era (Fig. 8). Terrace and talus deposits overlie these bedrocks. Green schist which is hard and less susceptible to weathering is distributed from the riverbed to the left abutment, where the right abutment is dominated by quartz schist which is complexly sheared and weathered even in depth. The bed of quartz schist lies almost in the direction of the stream and dips normal to the abutment surface. The rock in the right abutment has many open joints exhibiting large permeability. The results of laboratory tests for green schist and quartz schist are summarized in Table 3.

(2) Instrumentation

In order to measure the deformation of the foundation and the change of permeability due to fill placement at B Dam, the extensometers and the boreholes for water pressure tests were installed at the right abutment. The location of the extensometers and the boreholes for water pressure tests is shown in Fig. 9. Four extensometers, D_t-1 , D_t-2 , D_t-5 and D_t-6 were installed beneath the inspection gallery, and two, D_t-3 and D_t-4 were beneath the filter zone. The measuring distance of each extensometer was 10m long. Boreholes for water pressure tests were drilled vertically at two points. Each borehole was 10m deep and water pressure tests were conducted with a single packer method.

(3) Results of Measurements

a) the relationship between the height of fill placement and the displacement of foundation

Fig. 10 shows the relationship between the height of fill placement and the displacement of the foundation. Fig. 11 illustrates the relationship between the height of fill placement and the strain of foundation which is obtained from dividing the displacement by the initial length of the extensometers. These two figures lead to the following:

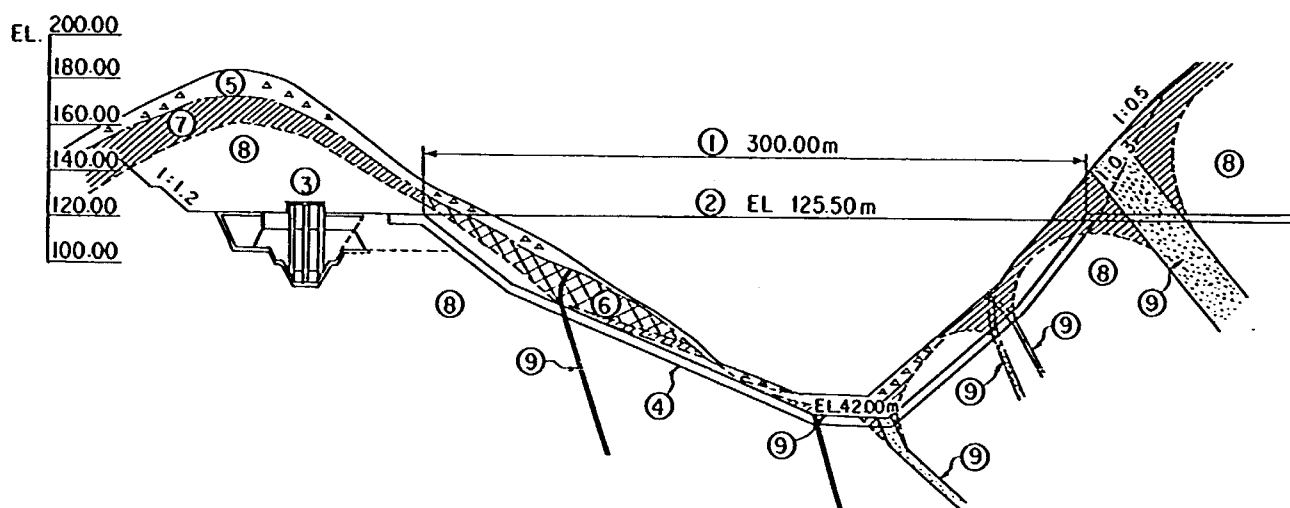


Fig. 8 Geological Map along Dam Axis (B Dam)

- | | |
|----------------------------------|----------------------------|
| 1. Length of Dam Crest | 6. Creep Zone |
| 2. Dam Crest | 7. CL-Class Rock |
| 3. Spillway | 8. CM-Class or Better Rock |
| 4. Gallery | 9. Fault or Fractured Zone |
| 5. Talus Deposit or D-Class Rock | |

Table 3 Results of Laboratory Soil Tests (Green Schist and Quartz Schist)

	Green schist	Quartz schist
Specific gravity	3.02	2.75
Water content	0.19%	0.08%
Effective porosity	1.01%	0.29%
Dynamic Poisson's ratio	0.34	0.33
Dynamic elastic modulus	$5.8 \times 10^5 \text{ kgf/cm}^2$ ($5.7 \times 10^4 \text{ MPa}$)	$4.6 \times 10^5 \text{ kgf/cm}^2$ ($4.5 \times 10^4 \text{ MPa}$)
Unconfined compressive strength	960 kgf/cm^2 (94.1 MPa)	750 kgf/cm^2 (73.5 MPa)

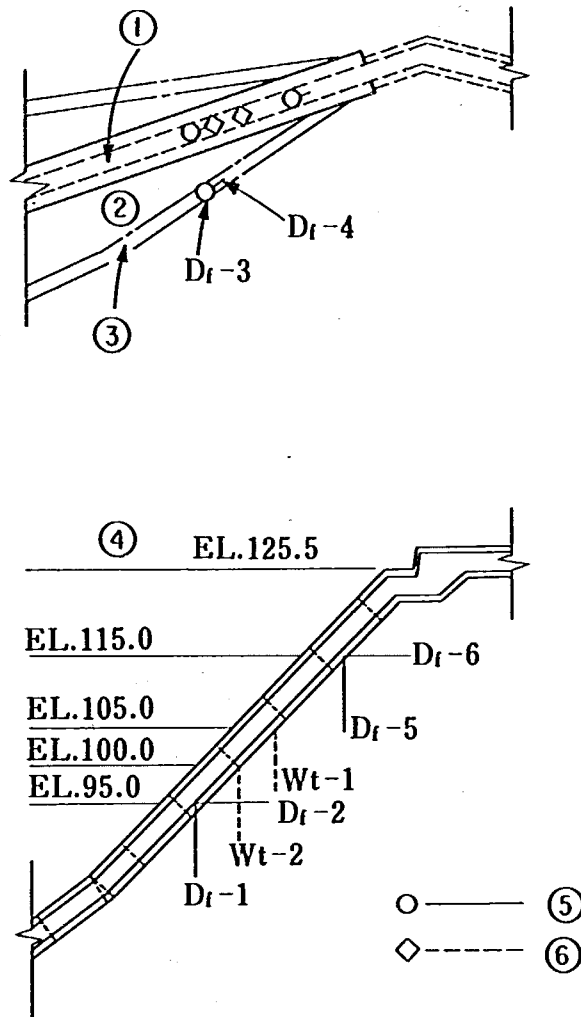


Fig. 9 Schematic View of Observations (B Dam)

- | | |
|----------------|-------------------------------------|
| 1. Gallery | 4. Dam Crest |
| 2. Filter Zone | 5. Extensometer |
| 3. Core Zone | 6. Borehole for Water Pressure Test |

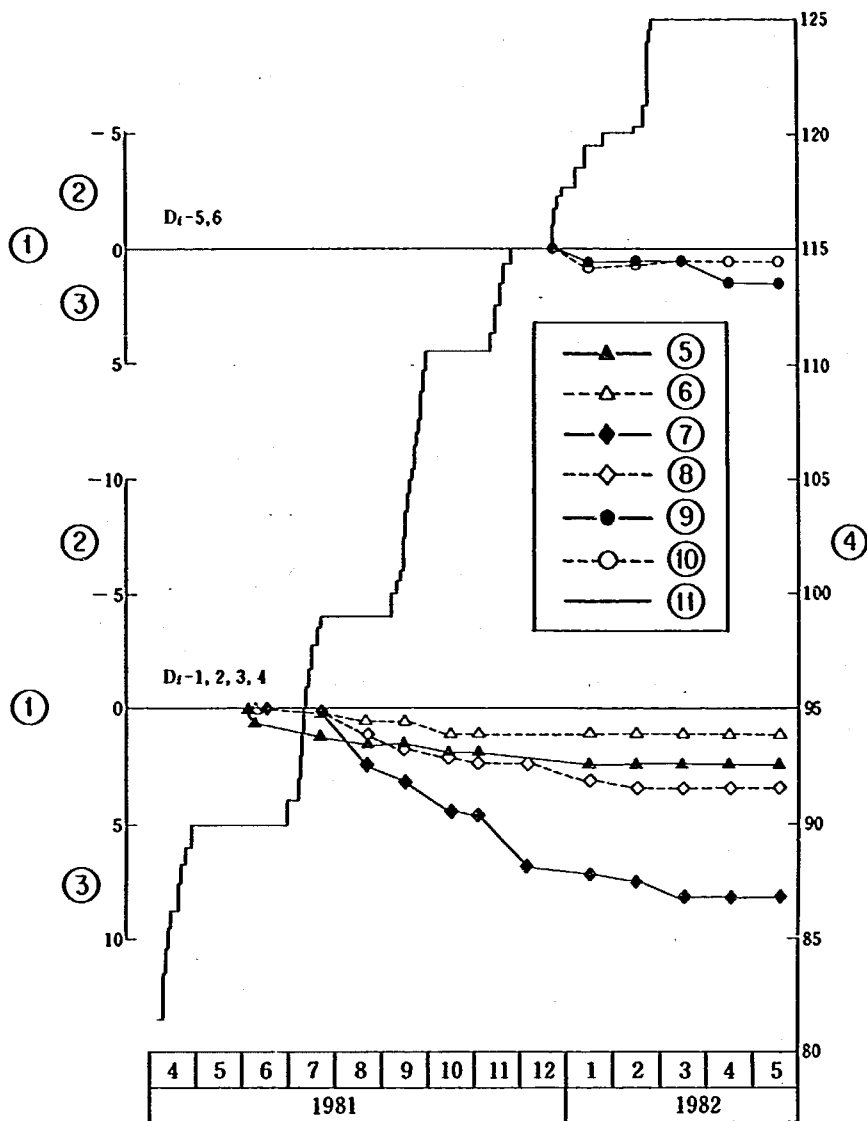


Fig. 10 Displacement of Foundation with Time (B Dam)

- | | |
|----------------------|------------------------------|
| 1. Displacement (mm) | 7. Df-3 |
| 2. Tensile | 8. Df-4 |
| 3. Compressive | 9. Df-5 |
| 4. Elevation (m) | 10. Df-6 |
| 5. Df-1 | 11. Height of Fill Placement |
| 6. Df-2 | |

1. Two pairs of extensometers were installed at about the same elevation. One was placed beneath the inspection gallery, and the other beneath the filter zone as shown in Fig. 9. The displacements measured by the former were smaller than those of the latter. This is the effects of the inspection gallery which replaced deformable foundation rock.

2. When the surface of fill placement reached the vicinity of the elevation of each extensometer, no tensile displacement was found at B Dam. However, if the measurements had begun earlier than the time indicated in Fig. 10, tensile deformation might have been measured as in A Dam.

3. The increase of vertical strain of the foundation ceases after fill placement progresses to some extent. In the case of A Dam, strain increased almost proportional to the loading (Fig. 5). Since the rock composing A Dam site is soft rock, the rock itself has a large deformability. However, the foundation rock in B Dam greatly increases in rigidity after the 23m overburden as shown in Fig. 11, because the deformability of schist in B Dam is derived from mostly open joints and because the rock itself has small deformability.

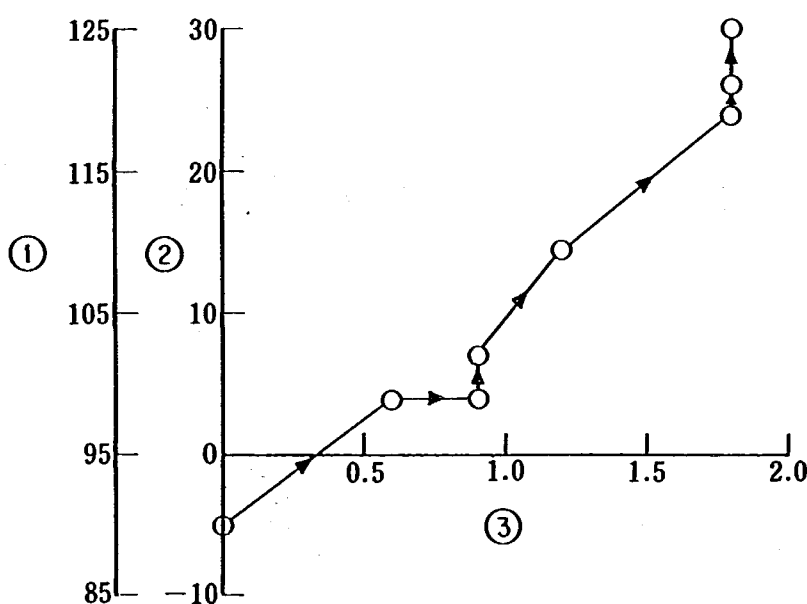


Fig. 11 Strain of Foundation due to Fill Placement (Df-1, B Dam)

- 1. Elevation (m)
- 2. Height of Fill Placement (m)
- 3. Vertical Strain ($\times 10^{-4}$)

b) the relationship between the height of fill placement and the change of permeability

The relationship between the height of fill placement and Lugeon value is shown in Fig. 12. This figure leads to the following :

1. Lugeon values measured at the testing hole Wt-2 is much larger than those measured at the testing hole Wt-1. However, in both testing holes, the Lugeon values tend to decrease due to fill placement. At the testing hole Wt-2, the critical pressure was recognized in the first test, resulting in fracturing of the material surrounding the borehole. Therefore Lugeon value measured in the second test is larger than that measured in the first one. However, the Lugeon values in this case also tend to decrease due to fill placement thereafter.

2. After fill placement exceeds about 5m, the Lugeon values almost converge. In the jointed rock of schist in B Dam, the consolidation effects on the permeability are limited. However, in the soft rock of A Dam, the permeability continued to decrease with the overburden loading.

c) the relationship between the strain in the foundation and the change of permeability

In B Dam, there were no extensometers at the same elevation with that of the borehole for water pressure tests. Therefore, the strain at the location of the borehole Wt-1 was estimated from the deformation of the extensometer Df-1. Fig. 13 illustrates the relationship between Lugeon values and the strain. This figure leads to the following :

1. Lugeon value decreases with the increase of compressive strain. This property is more evident in the results measured at B Dam than in those measured at A Dam which has a soft rock foundation.

2. While the strain is relatively small, the Lugeon values decrease very quickly. However, after the strain exceeds some value, the Lugeon values tend to converge.

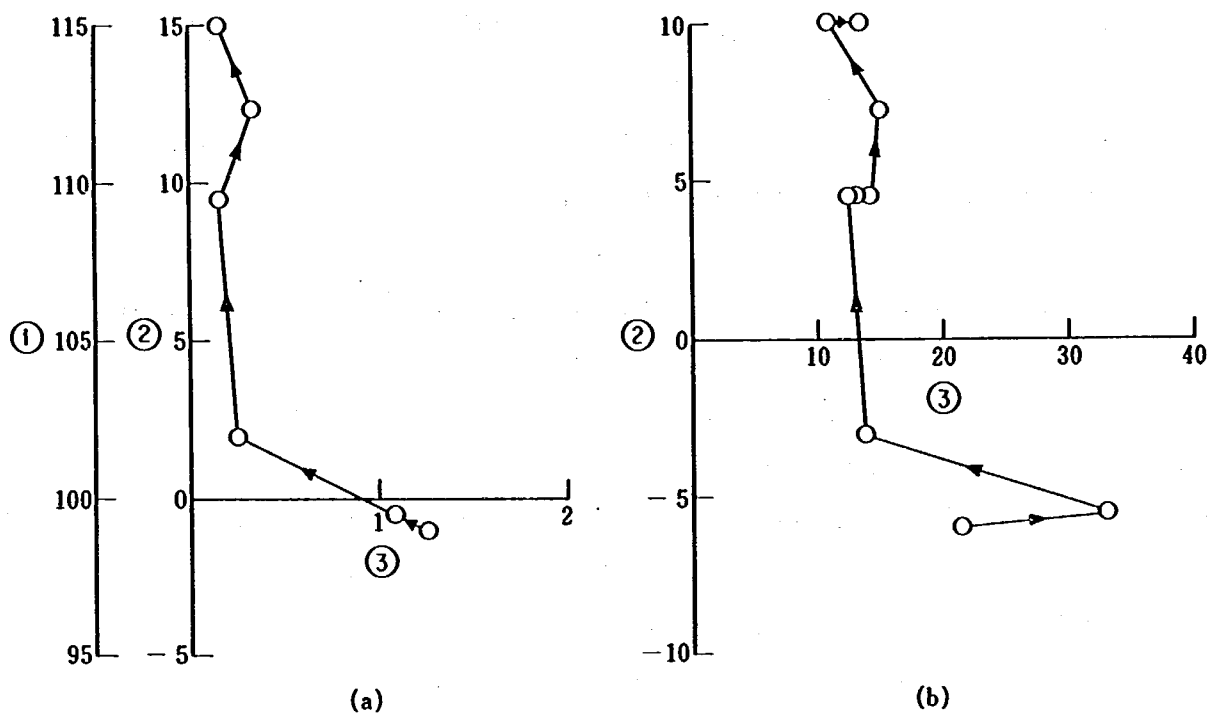


Fig. 12 Change of Permeability due to Fill Placement (B Dam)

- a. Wt-1
- b. Wt-2
- 1. Elevation (m)
- 2. Height of Fill Placement (m)
- 3. Lugeon Value

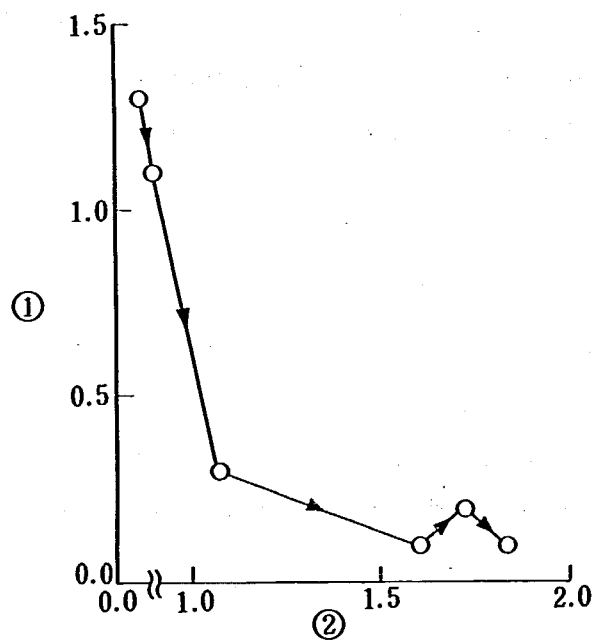


Fig. 13 Relationship between Lugeon Value and Vertical Station (B Dam)

- 1. Lugeon Value Observed at Wt-1
- 2. Vertical Strain ϵ_v ($\times 10^{-4}$)

4. THEORETICAL STUDIES

(1) Formulation of Mathematical Model

From the analysis of deformation and permeability measured at A Dam and B Dam, it was revealed that the rock foundation was consolidated due to fill placement resulting in decrease of permeability of the rock foundation. The authors try to explain the relationship between the displacement of the rock foundation and the change of permeability in this section by formulating the mathematical model.

Seepage water in the rock can be classified into two in accordance with the path of seepage. One is water which flows through the open joints (1), and the other is water which percolates within the pores of the rock itself. The authors hypothesize two mathematical models for seepage from the borehole. Model 1 corresponds to the seepage through joints, and Model 2 corresponds to pores.

Model 1 is shown in Fig. 14. The following assumptions are made for Model 1.

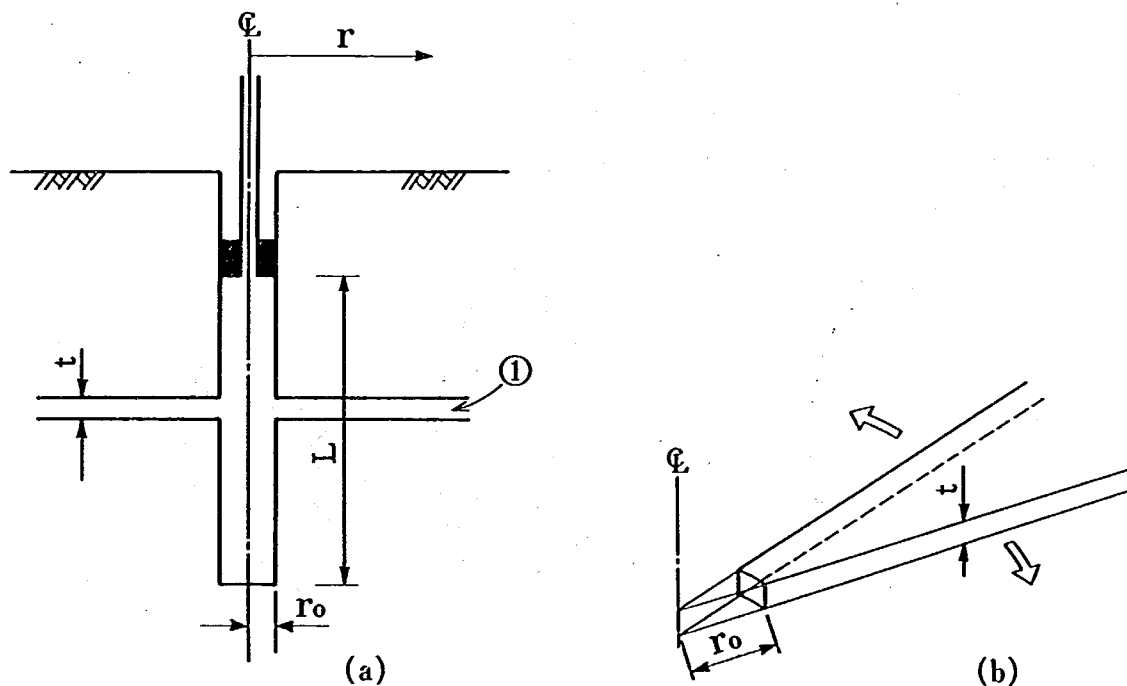


Fig. 14 Model 1

1. Joint

1. Just one joint crosses the testing hole. The joint is normal to the testing hole.
2. Joint is perfectly smooth.
3. Width of the open joint t is constant regardless of the radius r .
4. The rock is rigid and displacement of the rock foundation is only expressed by the opening or closing of the joint. The rock foundation deforms only in the longitudinal direction of the testing hole.
5. The rock itself is impervious and so water can only flow through the joint.
6. Flow through the joint is laminar within the region where r is smaller than the radius of influence r_1 . The radius of influence motioned here is equal to the distance from the center of the testing hole to the point where water pressure is assumed to be zero in obtaining the theoretical solution.

Assumption 6 should be explained in detail. There can be three types of flow in Model 1, as shown in Fig. 15. The Lugeon value is defined by the water take per hole per meter under water pressure of 10 kgf/cm^2 (980 kPa) obtained from the proportional part of injection pressure-injection quantity curve (hereafter, this curve is described as $P-Q$ curve) (Fig. 16). Of the theoretical solutions for the three types of flow patterns, the proportional relationship between P and Q can be proved only by adopting the flow pattern (a) in Fig. 15.

Model 2 is shown in Fig. 17. The following assumptions are made for Model 2.

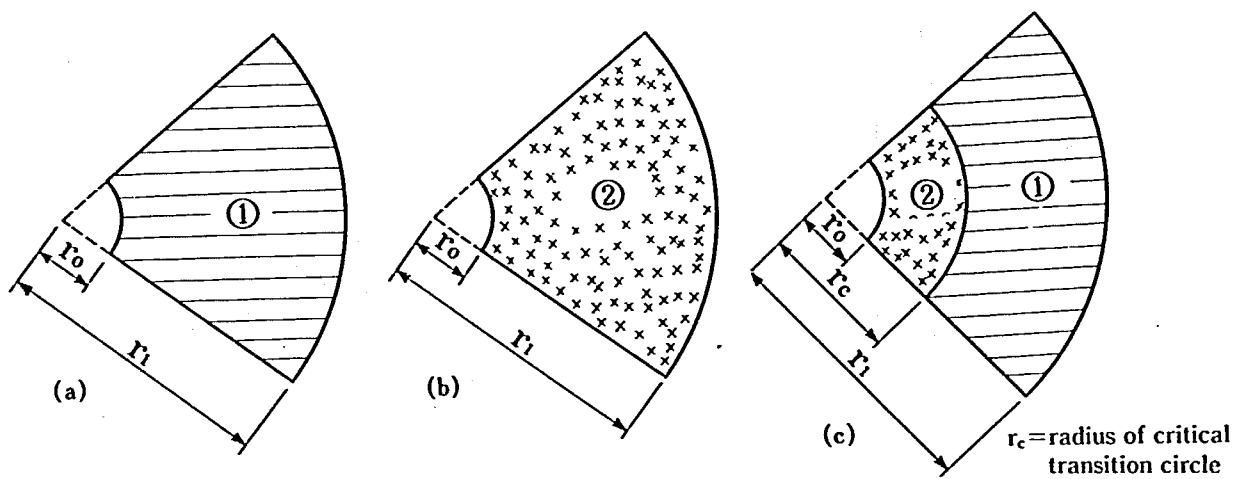


Fig. 15 Classification of Water Flow Conditions through a Joint

1. Laminar Flow
2. Turbulent Flow

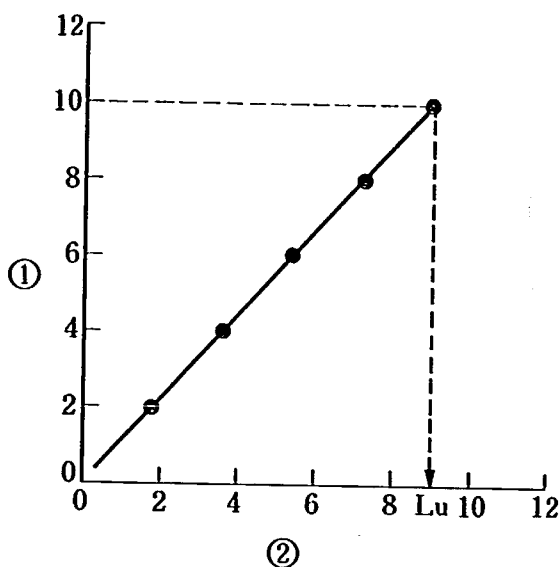


Fig. 16 P-Q Curve

1. Effective Pressure (kgf/cm²)
2. Rate of Water Injection (ℓ/min/m)

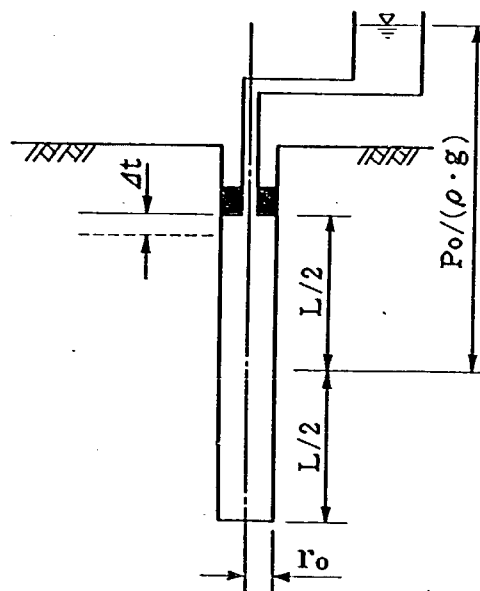


Fig. 17 Model 2

1. There are no joints in rock foundation and Darcy's law governs the flow in the rock foundation.
2. Rock foundation deforms only in the longitudinal direction of the testing hole. Displacement is related to the change of porosity of rock foundation.
3. The permeability coefficient of rock foundation is the function of only void ratio.

(2) Theoretical Solutions

The theoretical solution for Model 1 is obtained according to the following procedure.

The equation of equilibrium in a small element within a joint as shown in Fig. 18, may be written as

$$dP/dr = -2\tau/t \quad \dots\dots\dots (2)$$

in which, P = water pressure and τ = wall shear stress.

The wall shear stress τ can be obtained by the following equation:

$$\tau = \frac{\lambda \rho v^2}{8} \quad \dots\dots\dots (3)$$

in which, λ = resistance coefficient; ρ = mass density of water and v = velocity.

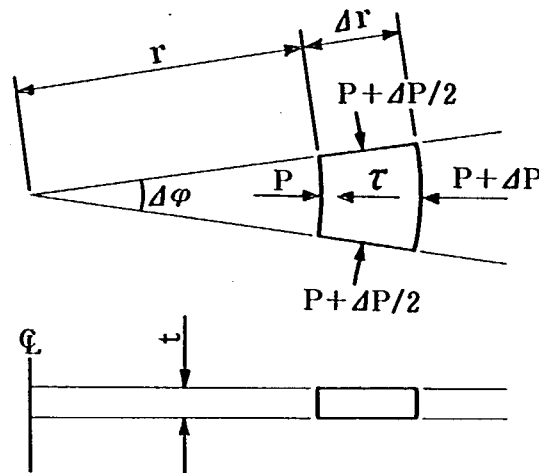


Fig. 18 Equilibrium of Force in a Small Element within a Joint

The equation of continuity gives us

$$\frac{Q}{2 \pi t} = r v = r_0 v_0 = r_1 v_1 \quad \dots\dots\dots (4)$$

in which, Q = discharge; v_0 = velocity at $r = r_0$ and v_1 = velocity at $r = r_1$.

Lomize, Louis et al. (2) have studied the coefficient of permeability for a open joint. According to their studies, in case the flow in a joint is laminar and a joint is perfectly smooth, the resistance coefficient λ is obtained from the following equation :

$$\lambda = \frac{96}{Re} \quad \dots\dots\dots (5)$$

in which, Re is Reynolds' number, which is

$$Re = \frac{D_h \cdot v}{\nu} \quad \dots\dots\dots (6)$$

in which, ν = kinematic viscosity of water and D_h = hydraulic diameter, which is

$$D_h = 4 R = 4 A / S \doteq 2 t \quad \dots\dots\dots (7)$$

in which, R = hydraulic radius; A = wetted cross section and S = wetted perimeter. Here, we notice that D_h is expressed only by t .

From the equations 2~7, the relationship between width of a open joint t and discharge Q is expressed as

$$\left. \begin{aligned} Q &= \frac{(P_0 - P_1) \pi t^3}{6 \mu \cdot \ell \ln(r_1 / r_0)} \\ t &= t_0 - \Delta t \end{aligned} \right\} \dots\dots\dots (8)$$

in which, P_0 = water pressure at $r = r_0$; P_1 = water pressure at $r = r_1$; μ = viscosity of water ($= \rho \cdot \nu$) ; t_0 = initial width of a open joint and Δt = compressive displacement of rock foundation.

Next the solution for Model 2 can be obtained according to the following procedure.

In case rock foundation is compressed Δt in the longitudinal direction of the testing hole, void ratio e_1 is

$$e_1 = e_0 (1 - \Delta t / L) - \Delta t / L \dots\dots\dots (9)$$

in which, e_0 = initial void ratio and L = length of the test section.

Taylor (5) studied seepage flow in soils comparing with flow in capillaries and obtained the relationship between the coefficient of permeability k and the void ratio e as follows.

$$k = C \frac{e^3}{1 + e} \dots\dots\dots (10)$$

in which, C is the function of unit weight of water, viscosity, grain size of rock foundation and so on. However, C can be assumed to be constant in this model.

Assuming the seepage in Fig. 17 to be the potential flow from point sources (6), we get

$$Q = \frac{2 \pi L P_0 k}{\rho g \cdot \sinh^{-1} \left(\frac{L}{2 r_0} \right)} \dots\dots\dots (11)$$

in which, g = gravity acceleration.

Equations 11~13 may be combined to yield the relationship between discharge Q and the compressive displacement of rock foundation Δt ,

$$Q = \frac{2 \pi L P_0 C}{\rho g \cdot \sinh^{-1}(L/2r_0)} \frac{\{e_0 (1 - \Delta t / L) - \Delta t / L\}^3}{1 + e_0 (1 - \Delta t / L) - \Delta t / L} \dots\dots\dots (12)$$

(3) Comparison of Measured Results and Theoretical Solutions

The authors compared the measured results with the theoretical solutions obtained by using the simple mathematical models for the relationship between the displacement of rock foundation and the change of permeability. The results measured in the testing hole Wt-1 of B Dam are investigated as examples. Input material properties used in the theoretical model are summarized in Table 4. The radius of the testing hole r_0 is 3.3cm and the influence radius r_1 is assumed to be 100m. Values of viscosity μ and density ρ of water are those at 20°C. P_1 is assumed to be 0 kgf/cm² (0 MPa) and P_0 is assumed to be 10 kgf/cm² (980 kPa) according to the method in obtaining Lugeon value. Discharge Q is obtained from the following equation:

$$Q \text{ (l/min)} = Lu \text{ (l/min/m)} \times L \text{ (m)} \quad \dots\dots\dots (13)$$

Furthermore, initial void ratio e_0 is assumed to be 0.1 considering the results of laboratory tests (Table 3).

Fig. 19 illustrates the theoretical solutions obtained from substituting the above-mentioned material properties into equations 8 and 12 and the measured results. Three theoretical curves for Model 2 with different porosities are also shown in Fig. 19. Fig. 19 leads to the following :

Table 4 Input Material Properties

r_0	0.033m
r_1	100m
P_0	10kgf/cm ² (980 k Pa)
P_1	0 kgf/cm ² (0 MPa)
μ	10.09×10^{-3} g /cm/s
ρ	0.99823 g /cm ³
g	980.0cm/s ²
e_0	0.1

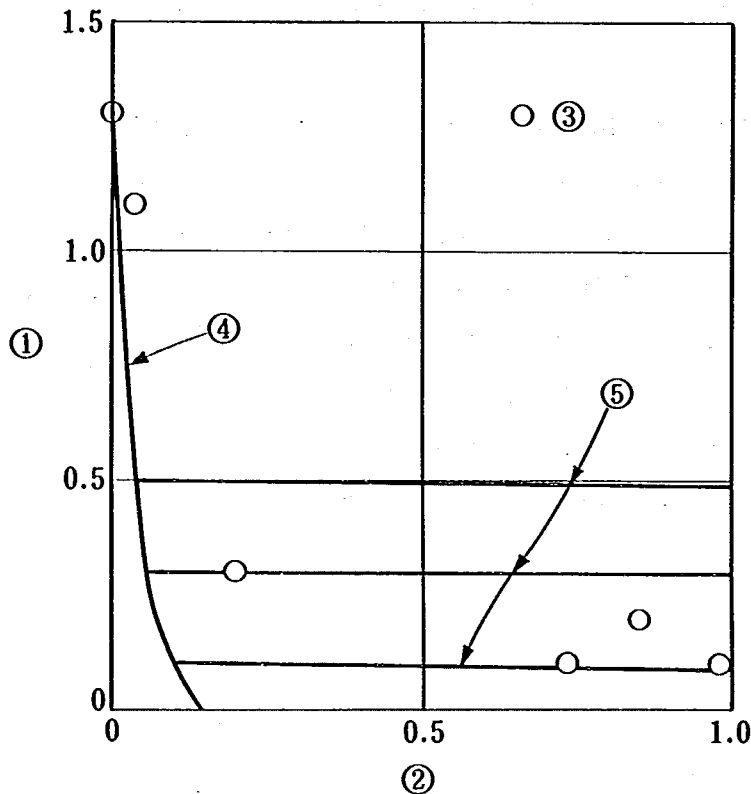


Fig. 19 Comparison of Measured Results and Theoretical Solutions

1. Lugeon Value
2. Compressive Displacement (mm)
3. Observed Results
4. Model 1
5. Model 2

1. The theoretical solutions obtained from Model 1 indicate the rapid decrease of Lugeon value with the increase of compression.

2. The theoretical solutions from Model 2 show that almost no effects on the decrease of permeability can be expected from consolidation.

3. If we consider that the flow around the testing hole Wt-1 changed from the flow expressed by Model 1 to that expressed by Model 2, the measured results in Wt-1, as shown in Fig. 19, can be explained by the combination of Model 1 and Model 2.

5. CONCLUSIONS

Fill placement works on the dam foundation as overburden loading. This loading influences the stress states in the foundation resulting in the change of strain and permeability in the underground. Therefore, the authors measured the deformation of the dam foundation and the change of permeability due to fill placement at two damsites. Furthermore, the authors tried to formulate the mathematical models to explain the relationship between the compressive strain of the foundations and the decrease of permeability due to fill placement. The results obtained from the measurements at two damsites and the theoretical studies lead to the following :

1. The results measured at A Dam which was constructed on soft rock foundation indicate that the tensile strain occurs temporally in the abutment due to fill placement and then it turns to compressive strain in accordance with the decrease of permeability.
2. The results measured at B Dam which was constructed on hard rock foundation also indicate that the fill placement causes the compression of dam foundation accompanying the decrease of permeability. However, the decrease of permeability occurs while the fill placement reaches to 5m, and does not occur afterwards.
3. The theoretical solutions obtained from the simple mathematical models well coincide with the measured results. Therefore, the authors feel that these theoretical models are useful for the effective grouting design for the foundation of fill dams.

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APPENDIX II.— NOTATION

The following symbols are used in this paper :

- A = wetted cross section ;
- C = constant ;
- D_h = hydraulic diameter ;
- e = void ratio ;
- e_0 = initial void ratio ;
- e_1 = void ratio after being compressed Δt ;
- g = gravity acceleration ;
- H = height of embankment ;
- k = permeability coefficient ;
- L = length of test section ;
- L_u = Lugeon value ;
- P = water pressure ;
- P_0 = water pressure at $r = r_0$;
- P_1 = water pressure at $r = r_1$;
- Q = discharge ;
- R = hydraulic radius ;
- R_e = Reynolds' number ;
- r = distance from center of testing hole ;
- r_0 = radius of testing hole ;
- r_1 = influence radius ;
- r_c = radius of critical transition circle ;
- S = wetted perimeter ;
- t = width of a open joint ;
- t_0 = initial width of a open joint ;
- v = velocity
- v_0 = velocity at $r = r_0$;
- v_1 = velocity at $r = r_1$;
- ΔP = increment of water pressure ;
- Δr = increment of radius ;
- Δt = decrement of width of a open joint ;
- $\Delta \phi$ = increment of angle ;

γ_t = unit weight of material of dam body ;

γ_{xy} = shear strain ;

ϵ_x = horizontal strain ;

ϵ_y = vertical strain ;

ϵ_α = strain parallel to slope ;

ϵ_β = strain perpendicular to slope ;

λ = resistance coefficient ;

τ = wall shear stress ;

μ = viscosity of water ;

ν = kinematic viscosity of water ;

ρ = mass density of water ;

θ = angle of inclination of slope.

CONSOLIDATION EFFECTS ON FOUNDATION PERMEABILITY

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Abstract :

The fill placement works on the dam foundation as overburden loading. This loading would influence the stress states in the foundation resulting in the change of strain and permeability in the underground. Therefore, the authors actually measured the deformation of the dam foundation and the change of permeability due to the fill placement at two damsites. The measured results for both damsites indicate that the fill placement causes the compression of dam foundations accompanying the decrease of permeability. The authors feel that these results are very helpful for the determination of effective grouting design for the foundation of fill dams. Furthermore, the authors theoretically try to explain the relationship between the compressive strain of the foundations and the decrease of permeability due to the fill placement by using simple models.

Keywords : dam foundation, seepage control, permeability, deformability.

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