PARTICULAR TREATMENT OF GROUND FOUNDATION ON THE SOIL WITH HIGH WATER PRESSURE

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ABSTRACT

Gousho Dam is located East of Fujinami Dam in Fukuoka-ken, Japan. The foundation soil had over 0.2 MPa of high water pressure due to a highly permeable andesite layer with many open cracks. This layer is connected to the reservoir of Gousho Dam. Heaving and/or piping was considered because of the high water pressure and significant cracking in the andesite layer beneath Fujinami Dam. Therefore, the two ground treatments described below were performed to control the soil permeability. (1) To prevent the ground heaving, the influence of water pressure was shut off by building a grouted cutoff curtain perpendicular to the center line of the dam. The highly permeable andesite rock contained many cracks (Lugeon value (Lu) > 1,000) such that the grouting could not be performed by the top-down method. The cement grout could not stay at the right position by using top-down grouting and too much volume of cement seeped out into lower layers, therefore, the vertical curtain grouting was built up by bottom-up grouting method. This bottom-up grouting achieved the efficient improvement at the right position with adequate cement volume. (2) The other treatment to prevent the ground water piping was thick blanket grouting. The blanket grouting was performed to the depth of 20m although that is normally performed up to the depth of 5m to 10m. With those two types of grouting countermeasures, the risk of failure associated with heaving and/or piping due to the seepage fracture of ground water at the dam foundation was reduced and acceptable factors of safety were achieved at Fujinami Dam.

INTRODUCTION

Fujinami Dam was built for water controlling against flooding, for the stabilization of water intake, and for the environmental conservation at the middle of Kose River. The location of Fujinami Dam is shown in Figure 1 and the outline of dam and the reservoir are shown in Table 1. The construction had started in 2002 and the dam body and the auxiliary facilities were finished in 2008. Test filling was started in 2009 and the dam has been in operation from 2010. At the beginning of the construction, there were concerns about the spring water from the surface excavation and the stability against the seepage flow around the riverbed center in the base excavation because the covering depth of pyroxene andesite layer which had artesian water with high pressure became shallower at the riverbed center. The layer continues from the reservoir of Gousho Dam,

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which was constructed previously in the upper tributary and located at East of Fujinami Dam (see Figure 3).

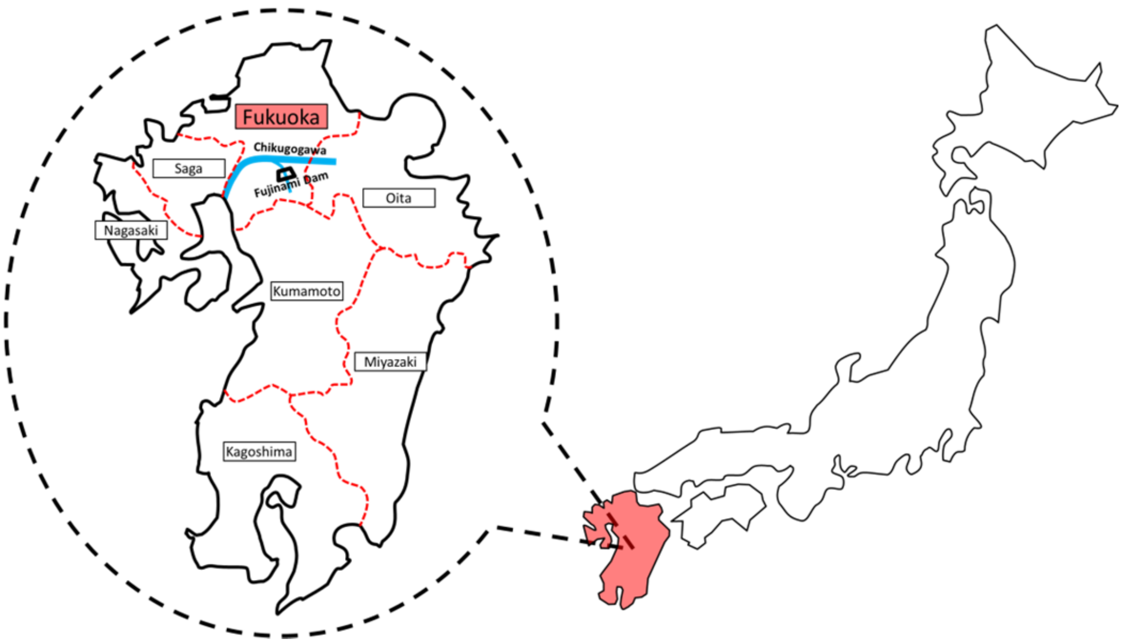

Figure 1. Location of Fujinami Dam

Table 1. Outline of Dam and Reservoir

Dam	Dam Type	Central Core Type Rock-fill Dam
	Height	52m
	Length	295m
	Volume	1,056,000m3
Reservoir	Watershed Area	21.7km2
	Flood Area	0.27km2
	Max Storage Volume	2,950,000m3
	Effective Storage Volume	2,450,000m3
	Lowest Water Level	EL118.5m
	Constant Water Level	EL123.0m
	Surcharge Water Level	EL135.5m
	Design Water Level	EL138.5m

CHARACTERISTICS OF SOIL

Fujinami Dam is located on mainly 4 soil and soft rock layers; muddy gravel (Mg), andesite pyroxene 1 (Ap1) and 2 (Ap2), and tuff breccia (Tb1). The cross-section of the layers are shown in Figure 2. The characteristics of those soil layers are shown in Table 2.

Table 2. Characteristics of Permeability of Each Soil

Mg	Per Lugeon test, the Lu is lower than or equal to 2 at the depth of 10m underneath riverbed and at the depth of 15m underneath right hand slope. The **critical pressure** correlates to the covering depth; the pressure is higher than 0.5MPa at the depth from 8m to 10m underneath riverbed, and at the depth from 15m to 20m.
Ap2	The Lu of the autobrecciated portion at the bottom of the lump part varies from 2 to 50. The consolidation index is similar to tuff breccias, and this soil doesn't have many cracks, therefore, the permeability of the bottom of Ap2 is lower than the lump.

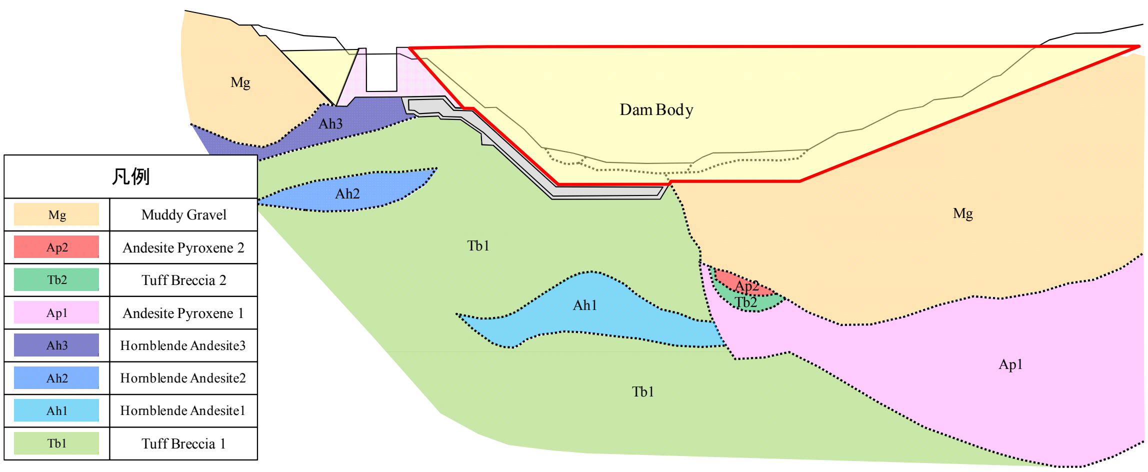

Figure 2. Cross-section under the Dam

THE SOIL STRUCTURE OF PRESSURING ON AP1

As stated in the former paragraph, Gousho Dam is located at the East side of Fujinami Dam (see Figure 3). The reservoir of Gousho Dam affects the ground water in the foundation layers of Fujinami Dam. Ap1 layer is connected from the bottom of the reservoir to underneath Fujinami Dam like a huge water tank. The lump part of Ap1 was assumed to be very permeable rock with many cracks. However, Ap1 and Ap2 are surrounded by the autobrecciated impermeable layers and the bottom surface of Mg has low permeability. Therefore, the ground water in the Ap1 layer is trapped and pressurized (see Figure 4).

Figure 3. Fujinami Dam (right) and Gousho Dam (Left)

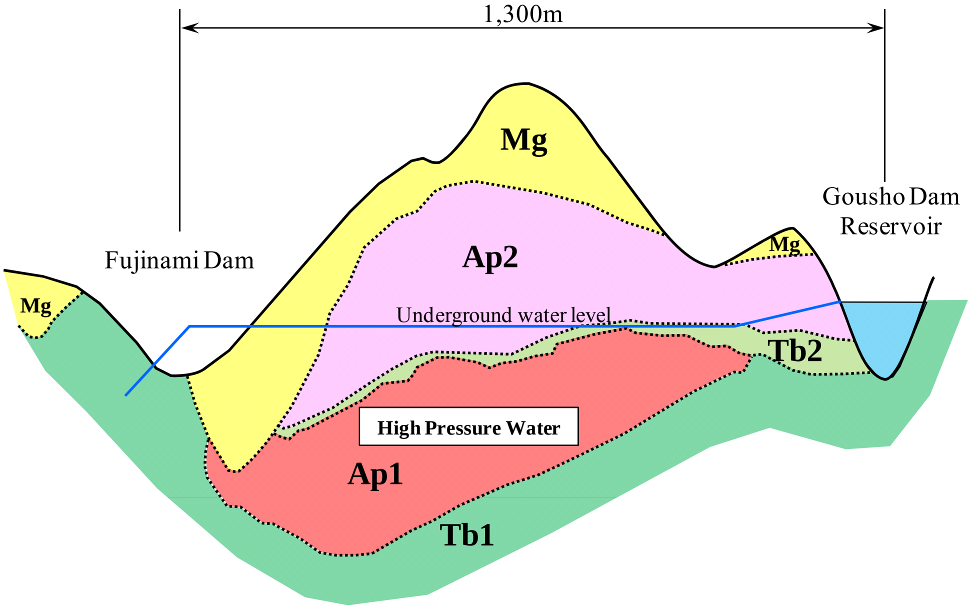

Figure 4. The model of underground structure

CONSIDERATION OF TREATING ARTESIAN HEAD

Acting Force on Mg before Excavation

Artesian Head. The artesian water head of Ap1 was EL. 124m during the reservoir filling test of Gousho Dam. No piping action in Mg layer was found from either the long-term observation of the water head or any site surface inspections.

Hydraulic Gradient. At the considered cross-section (see Figure 5) of artesian head (at No. 6+50), the maximum hydraulic gradient before foundation excavation was assumed as below.

$$I = \frac{h}{t} = \frac{26}{10} = 2.6 \tag{1}$$

h: Acting Head (=EL124m – EL98m = 26m)
t: Thickness of Covering Depth of Mg = 10m

The maximum hydraulic gradient of Mg from the laboratory piping test by boring cores at the riverbed on the Dam axis was greater than 30 and much larger than the on-site state.

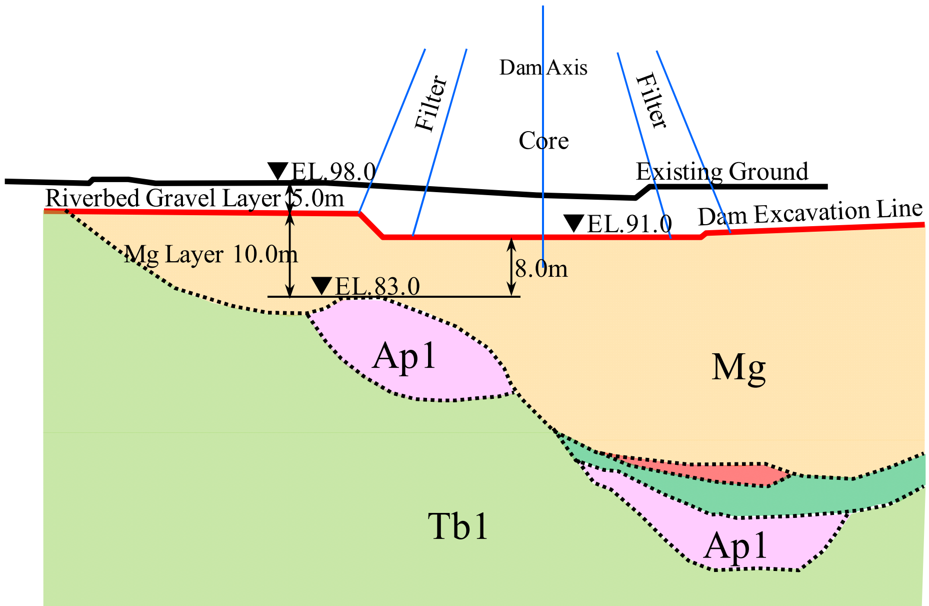

Figure 5. Considered Cross-section (No. 6+50)

Treatment of Artesian Water during Excavation

During excavation, the covering depth of Mg would become smaller. The loss of the covering depth would make the soil weight lighter and the hydraulic gradient larger. Therefore, these following issues were of concern;
(a) the uplift of dam foundation soil, and (b) the stability depression against the seepage by lowering artesian head. Therefore, the stability of dam foundation against artesian head in the excavation was examined. And the hydraulic gradient before excavation could be allowable since no piping action had been observed at the test filling of Gousho Dam.

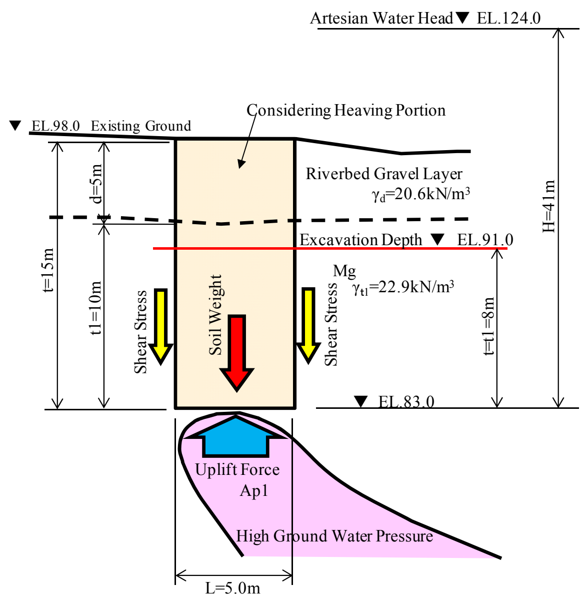

Figure 6. Heaving Model by Artesian Water Head

<u>(a) Uplift of dam base soil.</u> As shown in Figure 6, the heaving of dam base was examined considering the soil weight, shear stress, and uplift force. The shear stress before excavation was assumed as shown in the equations (2) and (3). The total of Soil weight and shear stress should be greater than uplift force.

$$(\gamma_{t1} \cdot t1 + \gamma_d \cdot d) \cdot L + 2Su \cdot t > \gamma_w \cdot H \cdot L \tag{2}$$

Thus,

$$Su > \frac{\gamma_w \cdot H - \gamma_{t1} \cdot t1 - \gamma_d \cdot d}{2 \cdot t} \cdot L \tag{3}$$

γ_{t1} : saturated unit weight of Mg (22.9kN/m^3)
γ_d : unit weight of riverbed gravel (20.6kN/m^3)
γ_w : unit weight of water (9.8kN/m^3)
t1 : thickness of Mg layer, d : thickness of riverbed gravel, Su : shear stress, L : width of considering soil chunk (this was determined by the top width of Ap1)

Su was assumed as 20.0kN/m^2 because no harmful phenomenon such as heaving had been observed at the test filling of Gousho Dam. The necessary lowering of artesian head during excavation was calculated by the equation (4) from the equation (2).

$$H < \frac{(\gamma_{t1} \cdot t1 + \gamma_d \cdot d) \cdot L + 2Su \cdot t}{\gamma_w \cdot L} \tag{4}$$

In the excavation, t = t1 = 8m, and d = 0, as shown in Figure 5. In order to satisfy the assumed shear stress (Su = 20kN/m^2), H < 25m. And, EL. 83m + 25m = EL. 108 m. Therefore, the artesian head should be lowered from EL. 124m to EL. 108m or lower to prevent heaving.

(b) Prevention of Stability Depression against Seepage by lowering artesian head. When the excavation bottom would have been EL. 91m, the covering depth of Mg would be 8m. In order to satisfy the hydraulic gradient before excavation i = 2.6, the artesian head h needed to be 21m by the equation (5).

$$i = 2.6 = h/t = h/8 \tag{5}$$

Therefore, the artesian head should be lowered from EL. 124m to EL. 112m to prevent the stability depression against seepage.

(c) Prevention of Stability Depression against Seepage by improving the permeability of Ap1. If the artesian head would not change and the permeability of Ap1 would be improved to about 10Lu ($1\text{x}10^{-4}$cm/s), the necessary improving depth was calculated to be 20m by the equation (6) if the permeability of Mg was $5\text{x}10^{-5}$cm/s.

$$h' = (\text{EL.124m-EL.91m}) = 33 = i \text{ x}8 + (5\text{x}10^{-5} / 1\text{x}10^{-4}) i \text{ x } d \tag{6}$$

Therefore, the necessary depth of Ap1 improvement should be 20m from the excavation bottom in order to prevent heaving.

In the actual construction of Fujinami Dam, both (b) and (c) methods were adopted in case of the damage of Mg by seepage or construction troubles. Lowering the artesian head (b) into ground level could be achieved by dewatering wells, but Ap1 layer was large enough to supply a huge volume of water, so the cutoff curtain grouting was provided by grouting. Reducing the permeability of Ap1 (c) into less than 10Lu was achieved by the blanket grouting to the depth of 20m from the excavation bottom line. Those countermeasures are shown in Figure 7.

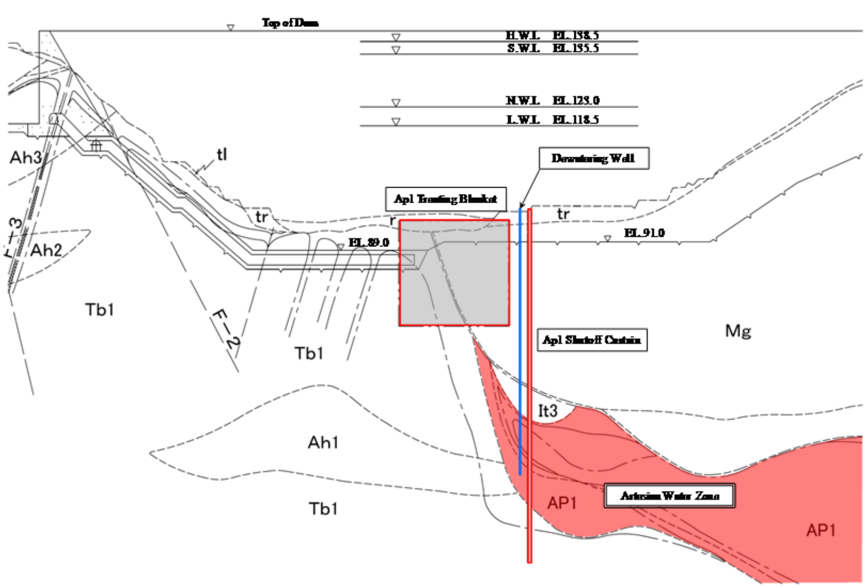

Figure 7. Particular Treatment Cross-section

CONSTRUCTION OF PARTICULAR TREATMENT

Ap1 Cutoff Curtain Grouting

In order to separate the pull up portion of Ap1 (surrounded by dash line in Figure 8) from whole Ap1 layer (hatching by pink), the curtain grouting was performed (red line).

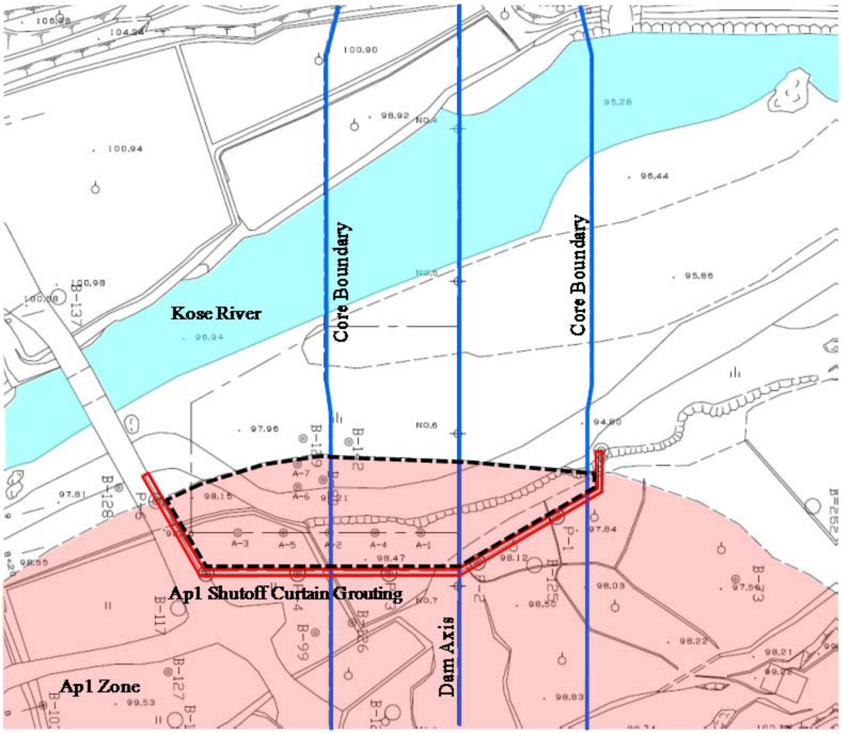

Figure 8. Ap1 Shutoff Curtain Grouting Plan

Since Ap1 layer has many cracks and high permeability, the normal grouting method of Top-down would lose too much cement volume into the Ap1 layer and the necessary portion was not able to be improved. Therefore, the Bottom-up grouting method was adopted to prevent the cement milk from flowing downward. These 2 types of grouting methods are shown in Figure 9. Based on laboratory tests performed prior to construction, the target value of ground improvement was less than or equal to 10Lu.

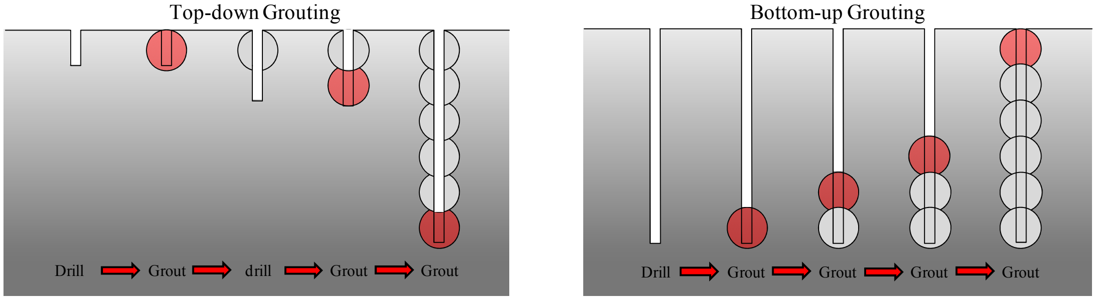

Figure 9. Grouting Method

The layout of grouting holes was multiple line layout as shown in Figure 10. The 1st and 2nd grouting holes were performed by Bottom-up method because of the reasons stated above. The 3rd and 4th grouting holes were performed by Top-down method because much of surrounding ground was already improved by the 1st and 2nd holes.

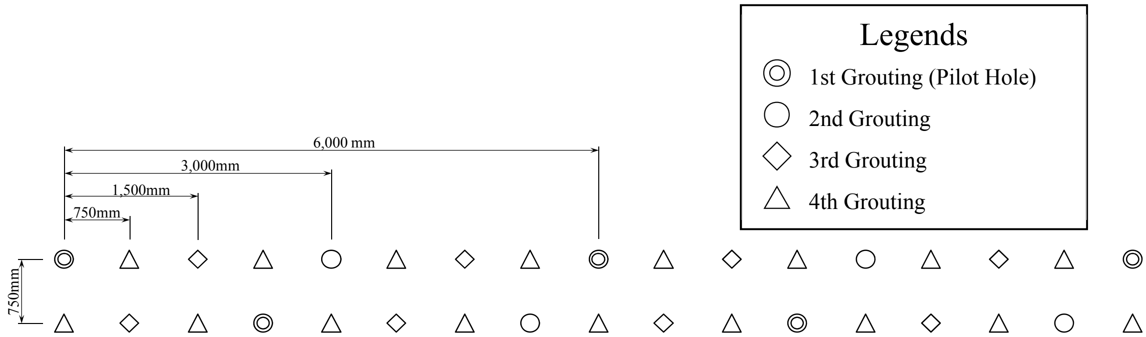

Figure 10. Ap1 Shutoff Curtain Grouting Layout

Ap1 Treating Blanket Grouting

The blanket grouting was performed as shown in Figure 11 in order to improve the impermeability of pull up portion of Ap1 layer.

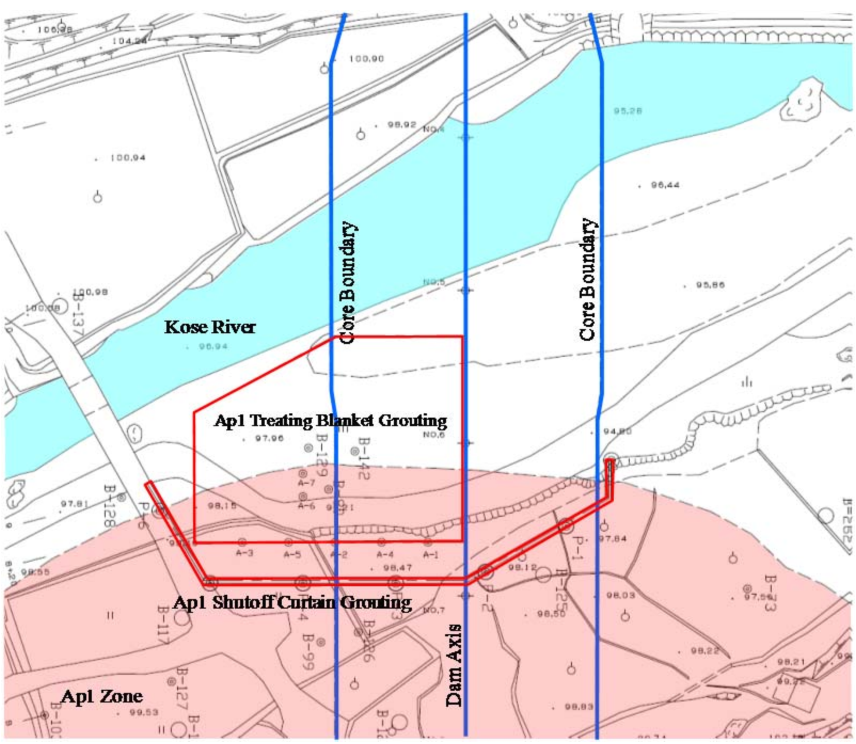

Figure 11. Ap1 Treating Blanket Grouting Plan

The target value of ground improvement was less than or equal to 10Lu, which is the same value as the shutoff curtain grouting. The grouting layout was 3m square mesh (from 1st to 3rd grouting) as shown in Figure 12 and the additional 4th or more groutings were performed at the center of the square mesh.

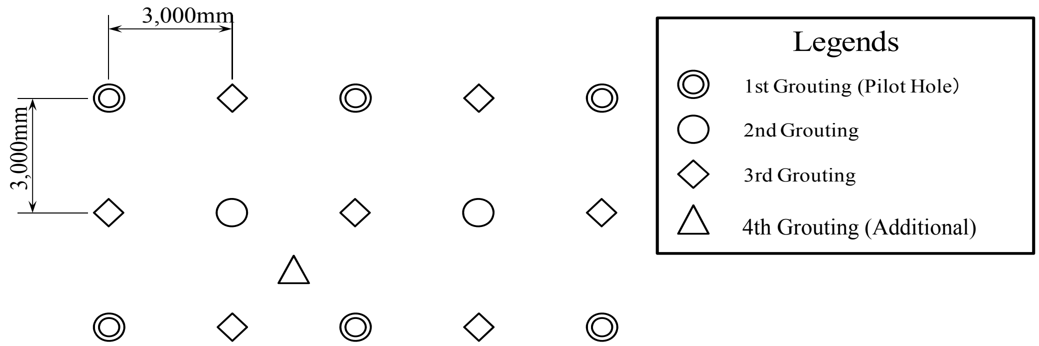

Figure 12. Ap1 Treating Blanket Grouting Layout

RESULT OF GROUTING

Ap1 Cutoff Curtain Grouting

At the beginning of the grouting, the artesian water of 200L/min to 500L/min was seeped from drilling holes and the drilling holes were collapsed. As construction continued, the artesian water was lowered and finally became 0 to 20L/min. The target of 10Lu was achieved at the final grouting stage. After the construction of the cutoff curtain grouting, dewatering wells were drilled and the artesian head was lowered to EL. 108m by dewatering 240L/min from separated Ap1 portion.

Ap1 Treating Blanket Grouting

Adjacent to the cutoff curtain grouting, the artesian water from the 1st drilling holes was about 200L/m. As the construction moved ahead, the water discharge became smaller and the water discharge from the final drilling hole was less than 1L/m. The target value of 10Lu was achieved after the 6th stage of additional grouting.

CONCLUSIONS

After finishing the construction of Ap1 treating blanket grouting, the dam foundation excavation was performed with dewatering wells located inside of the Ap1 cutoff curtain. As the difference in water head between the right and left side of the curtain grouting became larger, the seepage through the curtain grouting also became larger and there was concern that the gout curtain would be compromised. So the artesian head of both left and right side were observed and the dewatering volume was controlled to prevent the head difference from being too large. As a result, the dam foundation excavation was completed without any defects by the loss of covering depth of Mg.