第29回岩盤力学に関するシンポジウム講演論文集
(社)土木学会　1999年1月　審査付論文 (6)

Trial of Bentonite Grouting into the EDZ at AECL's Underground Research Laboratory

Kazuhiko Masumoto*, Yutaka Sugita**, Tomoo Fujita**, and Neil Chandler***

In the present sealing concept for disposal of the high-level radioactive wastes in Japan, the potential pathways for radioactive contaminant transport would be sealed in the closure phase by a combination of tunnel plug, backfill and grouting. The materials under consideration for the tunnel plug in these engineering barriers are a clay-based mixture and a concrete. The plug based on a bentonite material will be constructed to interrupt the migration of nuclide along the excavation damaged zone (EDZ) around the tunnel and to isolate the fracture zone. For these purpose, it is necessary not only to excavate a key for the plug but also to reduce the permeability of the EDZ adjacent to the clay-based plug. A bentonite grouting is one of the effective methods to reduce the permeability of fractured rock. A bentonite grouting trial was conducted at Atomic Energy of Canada Limited's Underground Research Laboratory for the purpose of developing a grouting procedure and the evaluation of grouting effectiveness for the EDZ around the clay-based plug. The results indicated that the bentonite grouting caused a reduction of permeability of the EDZ around the clay-based plug, although the EDZ at this site was very small and the initial permeability was very low.

Key words : High-level Radioactive Waste, Clay-Based Plug, Bentonite Grouting

1. Introduction

In the closure phase of the disposal facility of the high-level radioactive wastes, it is required to seal up the potential pathways of radioactive contaminants, which include the shaft or disposal tunnels used at the operation phase. In the present sealing concept in Japan, these potential paths should be closed by the combination of plug, backfill and grouting. The materials under consideration for the tunnel plug in these engineering barriers are a clay-based mixture and a concrete. The plug based on a bentonite material will be constructed to interrupt the migration of nuclide along the excavation damaged zone (EDZ) around the tunnel and to isolate the fracture zone, which has a high hydraulic conductivity compared with the intact rock. As these sealing functions are requested in the clay-based plug, it is necessary not only to excavate a key for the plug but also to reduce the permeability of the EDZ developing adjacent to the plug. A bentonite grouting is effective method to reduce the permeability of fractured rock, but the grouting into the EDZ is difficult because many of the fractures in the EDZ are connected with the excavation surface and cannot be filled efficiently by pressurizing the grout slurry.

This paper describes the results of a trial of a bentonite grouting into the EDZ developed around the clay-based plug. The grouting was carried out at a depth of about 395m in Atomic Energy of Canada Limited's (AECL) Underground Research Laboratory (URL).

2. Outline of the bentonite grouting trial

This grouting trial was conducted as one of the experiments of the Tunnel Sealing Experiment (TSX)

*　JSCE member　Kajima Corporation
**　JSCE member　Japan Nuclear Cycle Development Institute
***　nonmember　Atomic Energy of Canada Ltd.

being constructed in the granitic rock of the Pre-Cambrian Canadian Shield at a depth of about 420m (420 Level) in the URL (Fig.1). The main objective of the TSX is to examine the sealing performance of two full-scale bulkheads (a compacted clay-block bulkhead and a concrete bulkhead) under the high hydraulic gradient with the seepage through and around these plugs being monitored [1]. In the TSX, both bulkheads were keyed through the EDZ as an effective way to interrupt the flow along the EDZ. While, in the sealing concept, the bulkhead is mainly expected to act as a barrier against the flow through the tunnel backfill, the excavation of a key will also provide the cut-off of flow through the EDZ. Bentonite grouting was applied to the EDZ adjacent the TSX clay bulkhead as one of the techniques to increase the sealing function of the key (Fig.2). However it was difficult to evaluate only bentonite grouting in the TSX since the EDZ was interrupted by the combination of the key and the grouting. In those terms, the bentonite grouting trial was conducted in Room 419 excavated about 25m above Room 425 to estimate the sealing effectiveness of only the bentonite grouting into the EDZ. In Room419, a trial key, 1m deep by 2m wide, was excavated for the examination of the feasibility of construction of a square-shaped key in the TSX (Fig.3). The EDZ is induced both by the blasting during the excavation and by the stress redistribution around the tunnel [2),3)]. Room 419 has the same orientation and geometry as Room 425, therefore, it was expected that the stress distribution around Room419 was same as around Room 425 and the EDZ developing around the trial key had characteristics similar to the EDZ around the clay bulkhead in the TSX tunnel. The grouting trial was conducted under similar geometrical and geological conditions to the conditions that existed in TSX tunnel near the clay bulkhead.

The bentonite grouting trial was divided into two parts - development of a grouting procedure and evaluation of grouting effectiveness. The procedure was composed of drilling boreholes, grout injection and borehole filling. The purpose of filling the boreholes with bentonite cylinders after grouting was to prevent the grouting boreholes from acting as flow paths instead of reducing the permeability in the rock around these boreholes. A seepage test (also referred to as a connected permeability test) was conducted to estimate the effectiveness of grouting by comparing the seepage before grouting and after grouting.

3. Layout of the experiment

The in situ stresses in the rock at the depth of 420 m in the URL have been measured by various methods [4]. The back analysis, one of these methods, resulted that the three principal stresses were 60, 45, 11 MPa, with the maximum and intermediate principal stresses being sub-horizontal and the minimum principal stress being sub-vertical. As a result of the high ratio of horizontal to vertical stresses, the micro cracks induced by the tensile stress were developed in the rock in the sides of the tunnel, and cracking was induced by the compressive stress concentrations in the rock in the roof and floor of both Room 419 and Room 425. A few fractures have been observed in the key to depths up to 300 mm in the floor in the tunnels [3] and a seismic velocity survey using a Micro-Velocity Probe indicated about 100 mm of EDZ. Based on these results, the layout of the boreholes for grouting trial was designed as shown in Fig.3. The boreholes for grouting were 76mm-diameter with shallow inclinations. Boreholes G1, G2, G3 were inclined at 20° with a nominal length of 1.0 m and boreholes G4, G5 were inclined at 10° with a nominal length of 0.7 m. The shallow drilling angle allowed the borehole to intersect the EDZ for a longer distance, however, if the boreholes were too shallow, a large amount of grout slurry could be expected to leak into the tunnel. The collar of G2 was situated along the edge of the key at the location of the highest compressive stress concentration. The other boreholes were drilled around this borehole, collared on the vertical face of the key.

Fig.3 shows the concrete dam and the method of seepage collection for the connected permeability test. The concrete dam was 1.2 meter high and 0.2 meter wide. A distance of 0.2 m is similar to the extension of the clay bulkhead in the TSX beyond the edge of the rectangular bulkhead key. To prevent seepage directly along the concrete-rock interface, two lines of bentonite strips were placed across the floor under the dam on the grouting side and one line was placed under the dam on the opposite side. An overflow pipe was put at a height of 1.0m on the dam to keep a constant water level in the reservoir. To collect and monitor the seepage, V-shape trays were inserted into saw cuts on the face of vertical key and a plastic tub was placed below the bottom of this tray to accumulate the seepage. This tub and the rock face were covered with a plastic sheet to prevent condensation or evaporation at this site.

4. Grouting procedure

4.1 Drilling boreholes

Table 1 Results of Hydraulic Test

borehole No.	test interval (m)	Permeability (m/s)
G1	0.85	6.33E-13
G2	0.64	2.06E-12
G3	0.78	2.40E-11
	0.69	1.95E-12
G4	0.50	4.56E-14
G5	0.57	3.86E-14

Table 1 shows the test interval and the results from the hydraulic pulse test [5] in each of the five boreholes. This hydraulic test was a pulse recovery test, with a 15 cm-long mechanical packer placed into each borehole and a maximum hydraulic pulse of about 100kPa applied in the isolated interval. The test interval was between the end of borehole and the packer. G3 had two test intervals to compare the hydraulic conductivity of the deeper test interval with of the shallower one. The result was analyzed using a Ramey type curve matching technique [5]. The measured hydraulic conductivity was larger in boreholes G1, G2 and G3 than in G4 and G5, and less in the deeper intervals than in the intervals including the shallow portion of G3. These results show that the rock near the floor of tunnel had a higher permeability because of the fractures in the EDZ.

4.2 Grout injection

The layout of grout injection system was shown in Fig.3. A constant pressure pump was used to eliminate pressure pulses. Grout was injected into all five boreholes at the same time. The actual maximum pressure in this trial was set at 500kPa to avoid the possibility that the floor of the tunnel might break out when grout was injected, especially in the shallow boreholes. Grout injection progressed from a diluted slurry (0.2% by weight) to a concentrated one (8.0%). In theory, the diluted slurry will penetrate fractures having small apertures and each successive injection, with an increasing percentage of bentonite slurry, would reduce the overall permeability of the fractured rock. The injection was carried out at 0.2%, 0.5%, 1.0%, 2.0%, 4.0%, 6.0% and 8.0% (Table 2) bentonite by weight based on the results of the tests at Kamaishi Mine [6]. At each change of the proportion, the injection boreholes were flushed at a pressure of 100kPa to completely replace the previous slurry with the new slurry. When the electric conductivity of the slurry from the outlet valves (flushing slurry) became almost the same as that of the injected slurry, the full injection pressure started to be applied (Table 3). Injection pressure was slowly increased for 15-20 minutes to reach 500kPa and was then maintained at

Table 2 Results of Grout Injection

proportion	spent time to 500kPa	injection time at 500kPa	total injection
0.2%	47min.	4hrs.	53.2L
0.5%	20min.	3hrs.20min.	60.5L
1.0%	16min.	1hr.33min.	29.8L
2.0%	12min.	1hr.42min.	30.4L
4.0%	16min.	1hr.39min.	17.6L
6.0%	20min.	1hr.50min.	12.5L
8.0%	16min.	1hr.52min.	4.1L

Table 3 Electric Conductivity of Grout Slurry

proportion	inflow	flushing	outflow
0.2%	0.823	0.821	0.800
0.5%	0.836	0.836	0.807
1.0%	0.886	0.860	0.848
2.0%	1.002	0.974	0.966
4.0%	1.208	1.060	1.142
6.0%	1.388	1.227	-
8.0%	1.587	1.336	0.977
reservoir water	0.235	unit: mS/cm	

the maximum pressure for 2 to 4hrs.

Fig.4 shows the flow rate of the grout versus injection time for each proportion. Inflow rate was determined by the change of slurry height in the hopper of the pump. Outflow rate was measured at the collection tub for the connected permeability test during grout injection. This figure indicates that the outflow rate is almost same as the inflow rate. In other words, most of the slurry flowed out from the vertical surface of the key or from the interface between the dam and the floor of tunnel. Fig.5 shows the displacement between rock and dam during grout injection as measured by LVDTs (Linear Variable Displacement Transformers). The LVDTs were installed after the injection of 0.5% slurry. Negative displacement means the distance between rock and dam increases. This distance increased during pressurized injection of the grout, but the displacements didn't recover completely following the release of pressure. The displacements were recovered only when the packers were removed from the borehole collars. This indicates that the pressurized grout slurry pushed up the dam through the cracks extending from the rock around the packer to the bottom of the dam and flowed out from the opened interface between the dam and the rock.

5. Evaluation of grouting

5.1 Evaluation from grout injection

The effectiveness of grouting the EDZ can be estimated from the total volume of grout injected during grouting operations. The accumulated injection volume of grout for each proportion during the grouting trials is shown in Fig.6. The slope of the lines in this figure is almost equivalent to the injection rate. The slope increased when the proportion was increased from 0.2% to 0.5%, which might mean that the 500kPa pressure caused the extension of some fractures or separation of the interface between dam and rock. However, the slope stayed nearly the same for the 0.5%, 1.0% and 2.0% injection proportions. The slope became remarkably smaller at proportions of more than 4.0%. This indicates that the flow paths were filled with bentonite grout at injection proportions of 4.0% or higher. In the Fig.4, it can be seen that the flow rate was gradually decreasing with the injection time, particularly at the higher proportions, because of the filling of fractures during the injection of one proportion.

The bentonite injection rate, inflow rate (mL/min) times proportion of grout slurry(%), is often used to evaluate the efficiency of grout injection, however, this indicator is only useful when all the grout slurry is injected into the rock. In this trial, the bentonite injection rate didn't reflect the actual efficiency of grout injection because of the large amount of measured outflow from the rock during grout injection. The outflow rate was slightly less than the inflow rate except at the injection proportion of 8.0%. Also the electric conductivity of the outflow was lower than that of the inflow (injected slurry) in all injection proportions (Table 3). It is possible that the outflow included fresh water seeping from the reservoir, thus diluting the grout outflow and lowering the electric conductivity. The outflow rate is therefore defined by,

[outflow rate] = [inflow rate] - [infiltration rate into rock] + [seepage rate from reservoir]

In this equation, the infiltration rate into the rock (mL/min) times the proportion of grout slurry (%) is equivalent to the efficiency of grout injection.

The seepage rate directly from reservoir during grout injection can be calculated before estimating the infiltration rate. The calculation of seepage is based on the conductivity difference between the inflow and outflow grout slurry and the water in reservoir. Because the water from reservoir had a very low conductivity compared with the grout slurry (Table 3) since the correlation between electric conductivity and the concentration of the grout slurry is nearly linear, it was possible to calculate the rate of seepage directly from the reservoir. The total measured outflow during grouting was then separated into reservoir seepage rate and grout outflow rate. In Fig.7, the measured outflow, which were measured after grouting at 0.2%, 1.0%, 4.0% and 8.0%, and the calculated seepage rate from the reservoir is plotted for each grout injection proportion. The seepage rate at grouting of 6.0% could not be calculated due to lack of data of electric conductivity in the outflow. The calculated rate indicates almost the same magnitude and trend as the actual measured rate, which increases from 0.2% to 0.5% grout proportion and has a peak at 0.5%. As noted earlier, this increase is probably resulting from the extension of some fractures or separation of the interface between the dam and the rock and the subsequent decrease in seepage rate is resulting from filling of the fractures or the interface. At 8.0% injection proportion, the calculated seepage rate is much larger than the measured rate because of the very low electric conductivity of outflow. To explain this, the filtering in the rock around the boreholes might be taken into consideration at such a high proportion injection. The filtering causes most of bentonite to remain in the rock and only the water comes out, therefore the conductivity of the outflow is already very low before mixing with the water from reservoir.

Shown in Fig.8 is the bentonite infiltration rate (the infiltration rate based on the calculated seepage (mL/min) times the proportion of grout slurry (%)) as a function of grout injection proportion. This figure indicates that 4.0% was the most efficient proportion in this grouting trial.

5.2 Evaluation from seepage measurements

The results of the connected permeability test were used to evaluate the effectiveness of grouting. The rate of seepage from the reservoir decreased when the grout was injected indicating a reduction in the permeability of the EDZ around the edge of the key. Fig.9 shows the seepage rate after pouring water into the reservoir during the entire grouting trial and Fig.10 shows the summary of change in seepage rates for each activity. During the four days following filling of the reservoir, the seepage rate decayed gradually with time as the bentonite strips along the concrete-rock interface started to swell upon contact with water. The initial seepage, before drilling the boreholes, was 6.7 mL/min as measured on the tenth day after filling the reservoir. The disturbance of the hydraulic condition by each activities caused a temporary increase in seepage rate, but the seepage rate decayed and became nearly constant after each activity. After drilling the boreholes, the seepage rate increased to more than 20 mL/min resulting from cutting part of bentonite strips during drilling of G2 and G3. Prior to grouting injection, the seepage rate became stable at 6.5 mL/min, 11 days after drilling, in spite of any disturbance that may have resulted from the hydraulic pulse test in the boreholes. The seepage rate when all five packers on the borehole collars were inflated just before grout injection remained at 6.5 mL/min. This result suggested that drilling of the boreholes had little effect on seepage rates and that, prior to grouting, the bentonite strips that were cut had expanded and resealed the dam-rock interface. On the sixth day after grouting, the seepage rate was 5.3 mL/min, and filling the boreholes accelerated this decrease because of their absorption of seepage and sealing the grout boreholes due to swelling. Since the rate of seepage after grouting is less than the seepage rate before grouting, it can be concluded that the grouting activities, including filling boreholes, were successful in reducing the permeability.

6. Conclusion

A trial of bentonite grouting was conducted to reduce the permeability of the EDZ developing adjacent to the clay-based plug in Room419 at AECL's Underground Research Laboratory. Grout injection progressed from a diluted slurry (0.2%) to a concentrated one (8.0%) using a maximum injection pressure of 500kPa. The injection proportion of 4.0% bentonite by weight was the most efficient in this trial. A seepage test was conducted to evaluate the effectiveness of grouting the EDZ around the clay bulkhead. The seepage rate from the reservoir through the rock near the floor of tunnel decreased after grouting activities. This result indicated that grouting resulted in a reduction of permeability of the EDZ adjacent to the clay-based plug. Although the EDZ at this site was very small (about 10cm in thickness based on observations and a seismic velocity survey) and the permeability was originally very low (10^{-11} to 10^{-14} m/s), grouting was successful in reducing the rate of seepage.

This grouting trial was conducted as a part of the TSX conducted by AECL with funding from JNC (Japan), ANDRA (France) and WIPP (USA). The authors would like to acknowledge the help by Mr. Paul Thompson, Mr. Edward Kozak and all technicians at the Underground Research Laboratory.

Reference

1) Chandler N., D. Dixon, Gray, M., Hara, K., Cournut, A. and Tillerson, J. : An in situ Demonstration of Technologies for Vault Sealing (in printing), Proc. 19th Annual Conf. Canadian Nuclear Society, 1998.

2) Chandler, N., Kozak, E. and Martin, C. : Connected Pathways in the EDZ and the potential for flow along tunnels, Proc. EDZ Workshop, Int. Conf. Deep Geological Disposal of Radioactive Waste, Canadian Nuclear Society, p.25-34, 1996.

3) Read, R. : Characterization Excavation Damage in Highly-Stressed Granite at AECL's Underground Research Laboratory, Proc. EDZ Workshop, Int. Conf. Deep Geological Disposal of Radioactive Waste, Canadian Nuclear Society, p.35-46, 1996.

4) Chandler, N., Read, R. and Martin, C. : In situ Stress Measurement for Nuclear Fuel Waste Repository Design, in Aubertin, M., Hassani, F. and Mitri, H. (eds.), Rock Mechanics Tools and Techniques, p.929-936, Balkema, Rotterdam, 1996.

5) Ramey, H. J., Agarwal, R. G. and Martin, I. : Analysis of 'Slug Test' or DTS Flow Period Data, Jour. Can. Petroleum Technology, vol.14, p.37-47, 1975.

6) Sugita, Y. and Fijita, T. : Clay Grouting Experiment at the Kamaishi Mine, Proc. Symposium on Rock Mechanics, p.276-280, 1996.

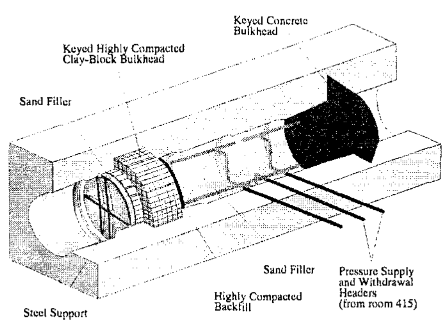


Fig.1 Outline of Tunnel Sealing Experiment

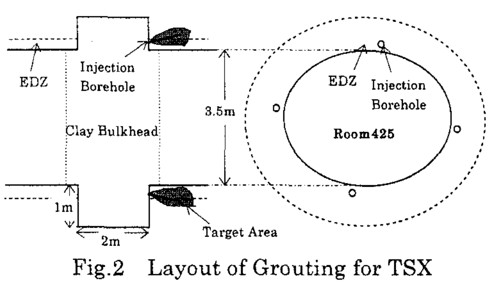


Fig.2 Layout of Grouting for TSX in Room 425

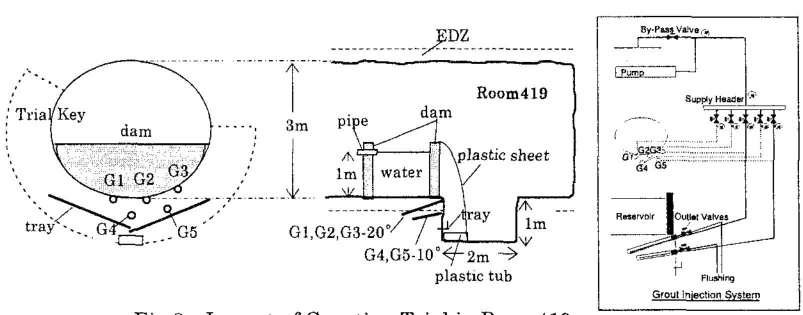


Fig.3 Layout of Grouting Trial in Room419

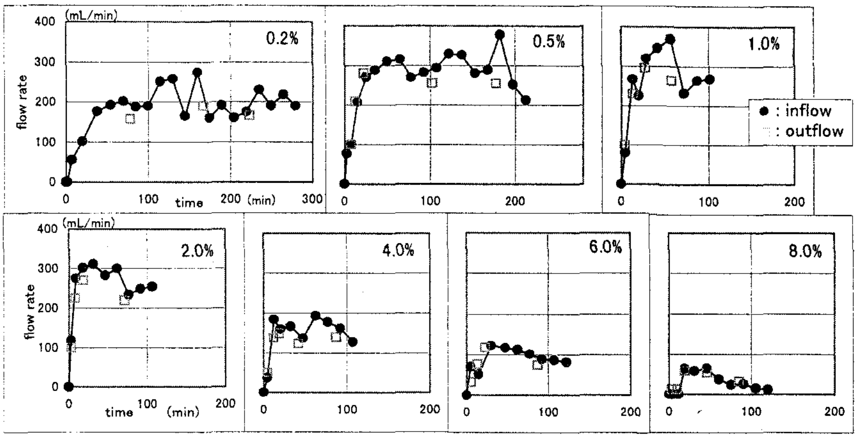


Fig.4 Result of Grouting Injection (inflow and outflow rate)

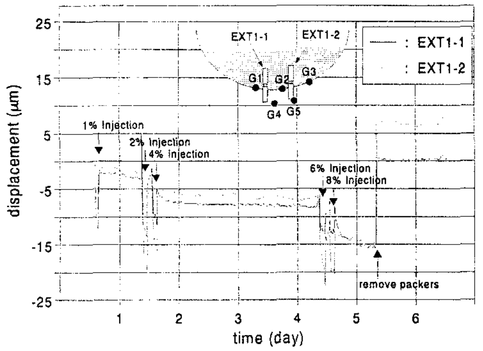


Fig.5 Result of LVDT Measurement

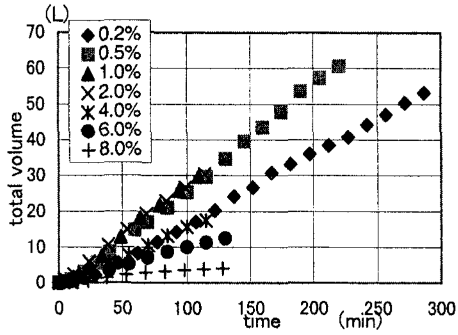
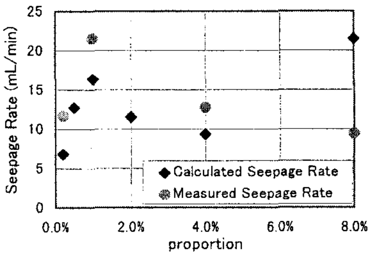


Fig.6 Result of Grout Injection (total injection volume)



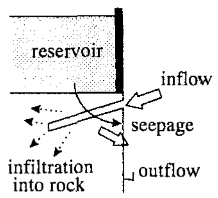


Fig.7 Seepage Rate from Reservoir

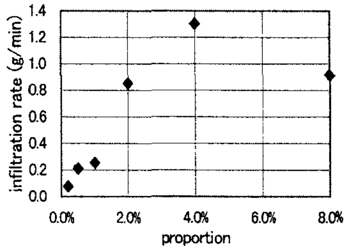


Fig.8 Estimated Bentonite Infiltration Rate

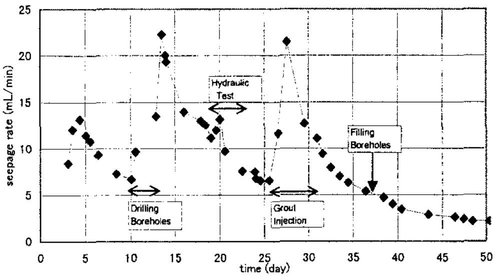


Fig.9 Seepage Rate during Grouting Trial

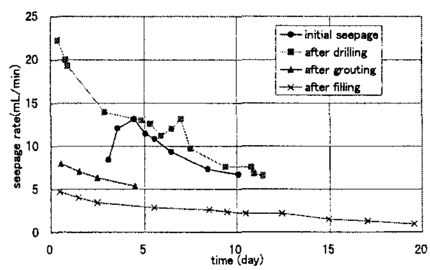


Fig.10 Seepage Change after Each Activity