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SYSTEM FOR ESTIMATING COEFFICIENTS OF PERMEABILITY IN REGIONAL GROUND WATER SURVEY

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In a regional ground water analysis by using a quasi three-dimensional finite-element method or the like, it is important to determine the spatial distribution of transmissivity *T*. Developed in the present study was a system for estimating the coefficient of permeability based on the density measured in situ with a split-spoon sampler with built-in tube. The water levels determined by numerical analysis based on the permeability coefficients estimated by the system were compared with measured water levels to ascertain the validity of the system.

Key Words : *N-value, split spoon sampler with built-in tube, density, gravel, coefficient of permeability, ground water survey*

1. INTRODUCTION

It was planed to draw ground water for industrial purposes in the Kumayama Industrial Complex (see **Fig. 1**) in Kumayama-cho, Okayama Prefecture. The required pumping discharge was several thousand cubic meters per day. It was anticipated that the pumping would influence the existing wells around the industrial complex. Under the circumstances, it was decided to carry out a regional ground water survey and analysis and determine the optimal location of the well in the industrial complex and the pumping discharge. The ground in this area is constituted by strata which consist mainly of gravel and has aquifers capable of feeding more than the required pumping discharge. However, because the transmissivity *T* of the

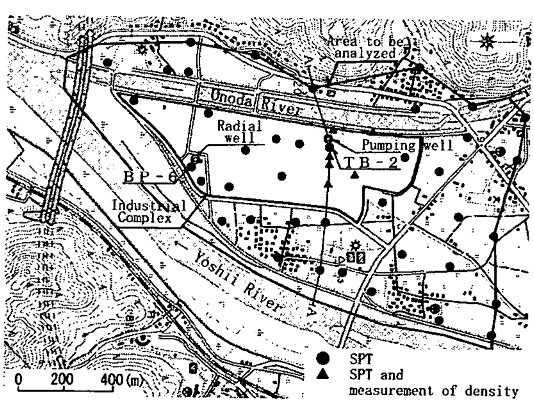


Fig. 1 Site and Area

ground varies from place to place, the major problem was how to grasp the distribution of transmissivity and how to evaluate the effects of

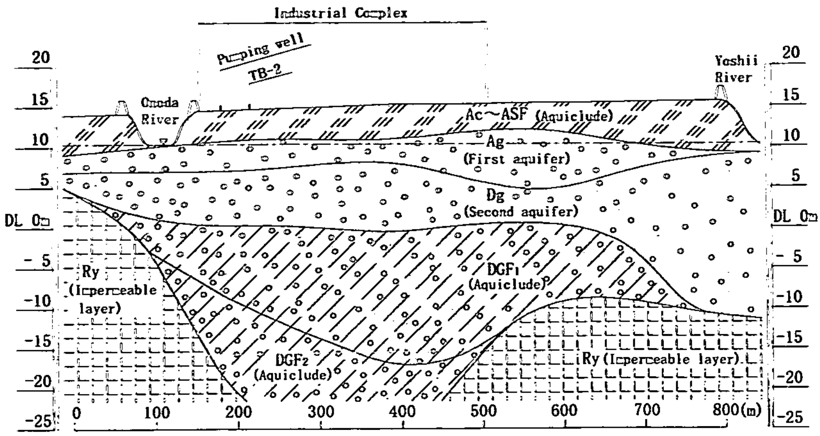


Fig. 2 Geological profile (A-A')

pumping on the neighborhood.

The distribution of transmissivity in an aquifer may be determined by changing the transmissivity arbitrarily so as to make the calculated water level equal to the measured one in the initial water-level analysis or by carrying out an inverse analysis probabilistically. In this study, however, the coefficient of permeability k and the thickness D of the aquifer were found though preliminary surveys to estimate the spatial distribution of transmissivity T. These parameters have the relation of $T = k \cdot D$.

To obtain highly reliable predicted values of behavior of ground water based on a sequential analysis, the coefficient of permeability k and the thickness D, in particular, of the aquifer have to be determined accurately. This accuracy requirement on the thickness of the aquifer can be met by increasing the number of survey points. The coefficient of permeability can accurately be determined by, for example, pumping tests, but the approach would be impractical from an economic point of view.

Accordingly the authors developed a system to estimate the coefficients of permeability k at dozens of points other than a few pumping test points of an aquifer economically by using the density, the N -value, and the grading measured with a double-tube-type standard penetration tester, or split-spoon sampler with built-in tube [1]. Then, a quasi three-dimensional finite-element seepage analysis was performed to verify the distribution of transmissivity T determined by the above system [2]. The analysis suggested the validity of the system.

2. PROFILE OF SITE GROUND

Fig. 1 shows the survey points. The site is situated between the class-A, or first-class, Onoda and Yoshii Rivers and is an alluvial low land; i.e., a flood plain formed by the sedimentation of the Yoshii River.

(1) Configuration of strata

According to geologic surveys by boring TB-2 and BP-6 carried out in the vicinities of a pumping well and a radial well for pumping tests (see **Fig. 1**), the ground was formed by the complex sedimentation of gravel, sand, clay, etc. **Fig. 2** shows a geological profile taken along line A-A' of **Fig. 1**.

a) Ac-ASF layer

It is an alluvium 1~7 meters thick and consists of alternate sandy silt layers and gravel-containing sand layers.

b) Ag layer

It is an alluvium, 0–13 meters thick, of sand and gravel with an N -value of 10~50.

c) Dg layer

It is an diluvium , 1–21 meters thick, of sand and gravel with an N -value over 50.

d) DGF$_1$ layer

It is a diluvium, 0–23 meters thick, of brown sand and gravel with an N -value over 50.

e) DGF$_2$ layer

It is diluvium of greenish-gray sand and gravel with an N -value over 50.

f) Ry layer

It is bedrock of rhyolite.

The survey by boring was carried out in a dry season, and the water surface in the bores were near the boundary between the Ac-ASF layer and the Ag layer (3–6 meters deep). The water levels in the bores through the year are still being surveyed.

(2) Permeability of strata

Fig. 3 and **Fig. 4** show a typical result of the soil survey. Taking notice of the grading and the density having large effects on the coefficient of permeability, the authors defined the strata as described below. The density was estimated from the N-value, which is also an index of compaction of sand and gravel.

a) The Ac-ASF, DGF_1, and DGF_2 layers, of which the fine-grained fractions F_c were 20% or more, were regarded as aquicludes. The fine-grained fraction F_c means the percentage of the furnace-dried weight of the soil fraction through a 75-μm sieve to the dry weight of a soil sample. The Ry layer was considered an impermeable layer. These layers were all treated as impermeable layers in analyses.

b) The Ag layer, of which the F_c and the N-value were less than 20% and 50 or less, respectively, was designated as the first aquifer. In the first aquifer, although the F_c was constant, the N-value and hence the permeability coefficient varied among survey points.

c) The Dg layer, of which the F_c and the N-value were less than 20% and over 50, respectively, was designated as the second aquifer.

Fig. 5 shows the configuration of the first and second aquifers in three dimensions. Each aquifer is complex in sedimentation and its thickness varies. The uneven thickness of the aquifers as well as their uneven constants has effects on the behavior of the ground water. Therefore, it is important in a regional ground water survey to grasp the varying thickness of aquifers accurately.

3. DETERMINATION OF PERMEABILITY COEFFICIENTS BY PUMPING TESTS IN FIELD

Pumping tests were carried out at TB-2 and BP-6. There is a radial well in the vicinity of BP-6, and the well was used as the pumping well for the pumping test. The following problems appeared in determining the permeability coefficients of the aquifers based on the results of the pumping tests.

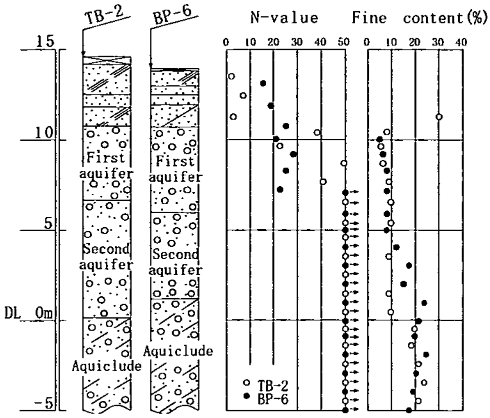


Fig. 3 Result of soil survey

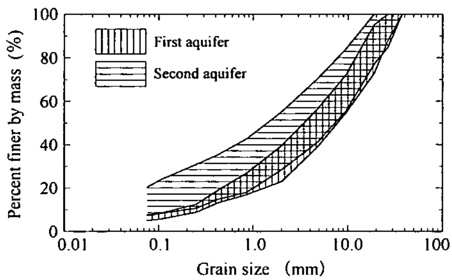


Fig. 4 Grain-size distribution

(a) As shown in **Fig. 7**, the aquifer is of an unconfined multiple type; accordingly, what kind of pumping test should be carried out? How should the coefficients of permeability be determined from test results? (b) The ground configuration around the TB-2 is very complex; accordingly, how should the thickness of the aquifers be estimated? (c) The radial well was used as the pumping well for the pumping test; accordingly, how would the construction of the well influence the results of the pumping test?

Under the circumstances, the permeability coefficients of the aquifers were determined in accordance with the flowchart of **Fig. 6**.

(1) Pumping test at TB-2

Fig. 7 shows the arrangement of the pumping well at TB-2 and observation wells. The pumping tests were carried out in the first and second aquifers to determine their coefficients of permeability k_f.

a) Results of pumping test in first aquifer

Fig. 8 shows the relation between the drawdown s and t/r^2 of each observation well while ground water was pumped up from the first aquifer through the pumping well at the rate of Q_w = 0.0486 m^3/min.

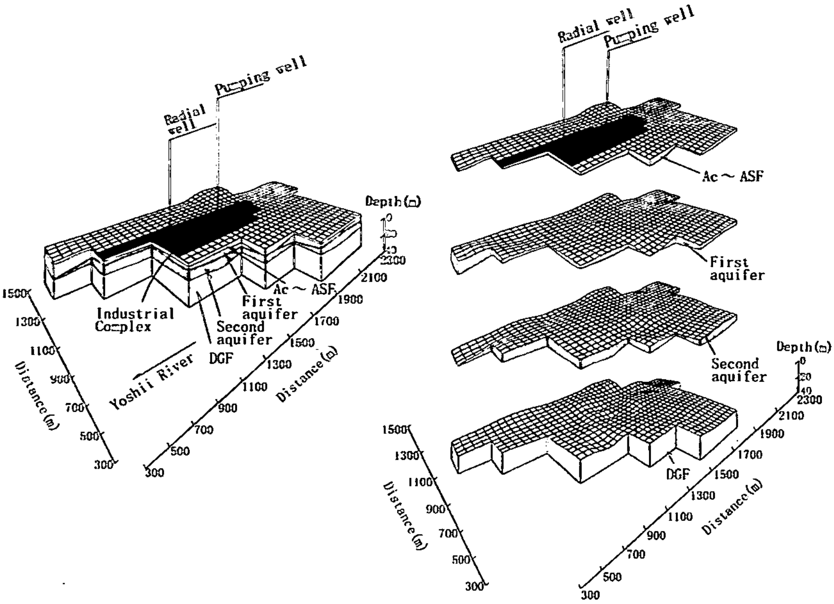


Fig. 5 Configuration of aquifers

The variable t is time; r, the distance between the pumping well and each observation well. As shown in **Fig. 8**, not all the data of each observation well align on a straight line. Each of the observation wells near the pumping well (r = 2–8 m) displayed a salient drawdown at an early stage of the pumping and thereafter its drawdown settled down to a certain slope. It seemed to have happened because water leaked from an upper layer to lower layer(s) while water was pumped up; accordingly, the coefficient of permeability k_f was examined by reproducing the result of the pumping test in an unsteady state by inverse analysis with the saturated/unsaturated seepage analysis program (UNSAF). The analysis revealed that the above phenomenon had been caused by a top layer with very high permeability (about one meter thick at DL = 9.0–10.0 m at TB-2) of the first aquifer around the pumping well [3]; therefore, the second-half straight-slope portion of the line representing the $s-(t/r^2)$ relation of each well in **Fig. 8** was regarded as indicating the coefficient of permeability of the first aquifer and Jacob's method[4] was applied to the straight-slope portions of all the observation wells to determine the average transmissivity T of the first aquifer as a whole at TB-2.

In Jacob's method, the relation among the transmissivity T, the coefficient of permeability k_f, and the thickness D of the aquifer is expressed by

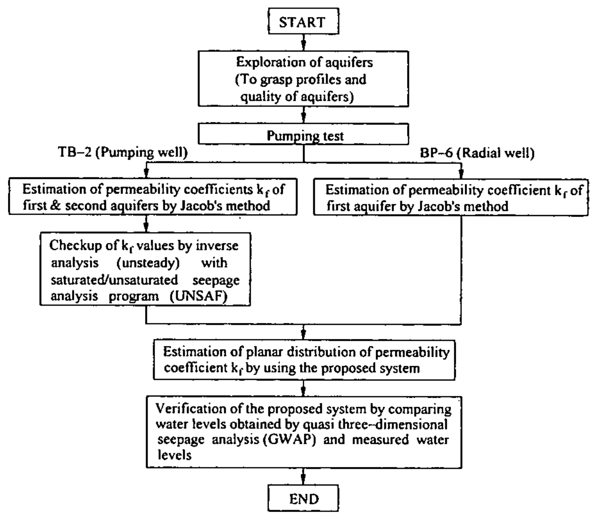


Fig. 6 Flow of analysis

the equation below.

$$T = k_f D = \frac{2.3Q_w}{4\pi\Delta s} \qquad (1)$$

where Q_w is pumping discharge; Δs, the difference in the drawdown per a logarithmic cycle of the exponent,on the straight-line portion in **Fig. 8**.

The above equation was applied to the $s-t/r^2$ relations of the observation wells to calculate their transmissivity T's, which varied widely from 24.3-67.4 cm^2/s. The variation was considered to have been caused by the different thickness and the different coefficients of permeability of the first aquifer among the different locations of observation wells. As mentioned earlier, the thickness D of the aquifers particularly around the pumping well varies largely. On the other hand, it was necessary to estimate the approximate coefficient of permeability of the first aquifer. Under the circumstances, although it might be a little crude, the average thickness 350 cm of the first aquifer was adopted.

Thus, the mean coefficient of permeability of the first aquifer at TB-2 k_{f1}^{TB-2} was found to be 1.3 × 10^{-1} cm/s.

b) Results of pumping test in second aquifer

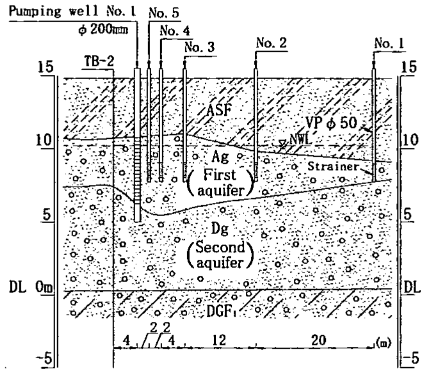
Fig. 9 shows the s $-t/r^2$ relation of each observation well while ground water was pumped up from the second aquifer through the pumping well at the rate of Q_w = 0.0635 m^3/min. As shown in **Fig. 9**, there is a tendency that observation data in the early stage of pumping are generally on a straight line, but the speed of drawdown lessened rapidly after some time of pumping. This phenomenon occurred because water leaked from the first aquifer to the second one whose permeability is poor. Therefore, the straight-slope portion of each line in **Fig. 9** corresponding to the early stage of pumping was considered to represent the permeability of the second aquifer, and the transmissivity of the second aquifer T_2^{TB-2} (= 3.3 cm²/s) was calculated based on the straight-slope portions [4]. The average thickness of the second aquifer was estimated to be 684 cm in the same way as the first aquifer.

Meanwhile, the rises of water levels toward the end of the pumping test shown in **Fig. 9** were due to the rises of water levels of the rivers which were caused by rainfall.

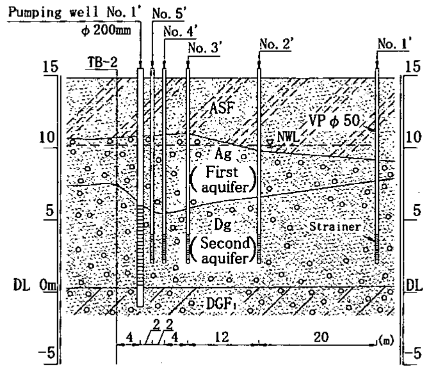
(2) Pumping test of first aquifer at BP-6

The pumping test of the first aquifer was carried out by using the radial well (by BP-6) as the pumping well. **Fig. 10** shows the arrangement of the radial well and observation wells.

Fig. 11 shows the $s-(t/r^2)$ relation of each observation well while ground water was pumped up at the rate of Q_w = 2.129 m^3/min. As shown in **Fig. 11**, all the observation data of each observation well generally aligned on a straight line. Without other applicable effective methods, Jacob's method was applied to the result of the pumping test to find the permeability coefficient k_f of the first aquifer, although the problem mentioned earlier of the radial well's being used as a pumping well remained.



(a) First aquifer



(b) Second aquifer

Fig. 7 Arrangement of pumping and observation wells (TB-2)

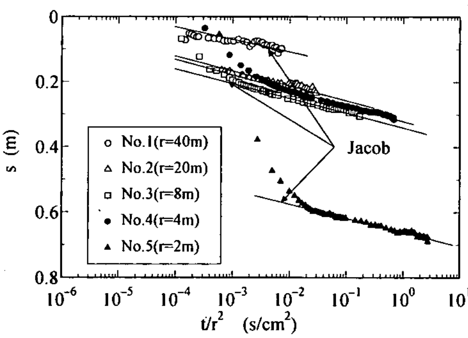


Fig. 8 $s-(t/r^2)$ relation (TB-2, first aquifer)

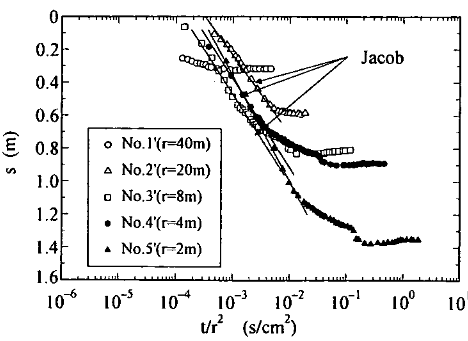


Fig. 9 $s-(t/r^2)$ relation (TB-2, second aquifer)

The mean transmissivity of the first aquifer T_1^{BP-6} was found to be 336.9 cm^2/s in the same way as described in the above paragraph (1). The mean thickness of the first aquifer was estimated to be 344 cm. The mean coefficient of permeability k_{f1}^{BP-6} of the first aquifer in the vicinity of the pumping, or radial, well was found to be 9.8 × 10^{-1} cm/s.

4. ESTIMATING SYSTEM OF COEFFICIENTS OF PERMEABILITY

In this chapter, the authors propose a system to estimate the spatial distribution of the coefficient of permeability of an aquifer based on various ground and soil data obtained through pumping tests at a few points and standard penetration tests at dozens of points, and laboratory permeability tests of specimens.

(1) Principle of estimating system of coefficients of permeability

Fig. 12 shows the basic relations of factors of the estimating system The relation among N-value, relative density D_r, and void ratio e [5] shown in **Fig. 12 (a)** is used also in the practical design calculation in the sand compaction pile method [6]. Regarding the N-Dr-e relation, the following equation was proposed by Meyerhof [7] based on the experiment made by Gibbs Holtz.

$$D_r(\%) = 21\sqrt{\frac{N}{0.7+0.01\sigma'_v}} \tag{2}$$

where σ'_v is the effective overburden pressure (kPa).

The e-D_r relation is given by the following equation.

$$D_r(\%) = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100 \tag{3}$$

where $e_{\max}$ and $e_{\min}$ are maximum and minimum void ratios, respectively.

Fig. 12 (b) shows the e - ρ_d - k_l relation, ρ_d being the dry density, k_l being the laboratory permeability coefficient. The coefficient of permeability is influenced by grain-size distribution and particularly fine-grained fraction F_c as is shown in **Fig. 4**; accordingly, fine-grained fraction F_c was used as the index of grain-size distribution in this study. The permeability varies depending on the structure and stratification of soil; therefore, the

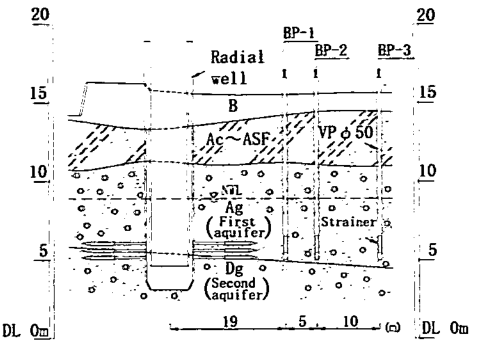


Fig. 10 Arrangement of radial well and observation wells(BP-6)

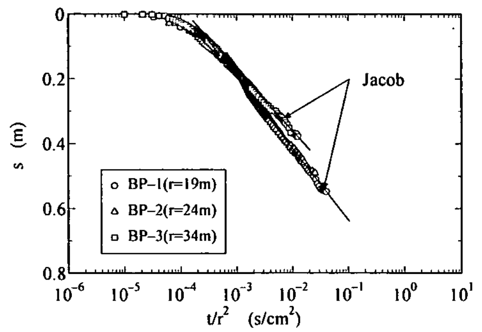


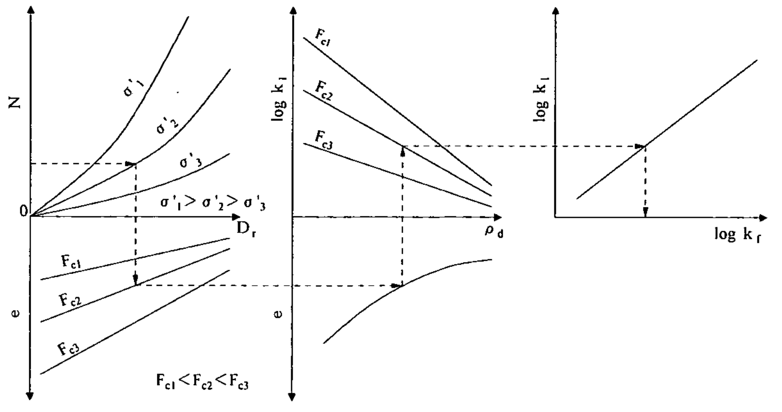
Fig. 11 $s-(t/r^2)$ relation (BP-6, first aquifer)

laboratory coefficient of permeability k_l of a soil with a void ratio differs from the field coefficient of permeability k_f of the soil with the same void ratio [8]. To raise the accuracy in estimating k_f, the k_l - k_f relation was ascertained as shown in **Fig. 12 (c)**.

By combining these three basic relations, k_f can be estimated from the N-value through the course shown by the broken line in **Fig. 12**.

In an actual site, the estimating system works as follows. The density of an aquifer is measured with a split-spoon sampler with built-in tube [1] at several survey points other than a few pumping test points and various laboratory tests are carried out to establish the basic relations of the estimating system. Then, surveys by boring (standard penetration tests and grain size analyses) are carried out at dozens of points to determine the varying thickness of the aquifer. Lastly, by applying the survey results, that is, N-values, F_c, and σ'_v to the basic relations of the estimating system, the spatial distribution of thickness and k_f of the aquifer are estimated.

Now, the method of measuring the density of an aquifer with a split-spoon sampler with built-in tube will be described briefly. As shown in **Fig. 13** and **Photo 1**, the split-spoon sampler with built-in tube consists of a standard penetration tester, and five



(a) $N-D_r-e$ relation (b) $e-\rho_d-k_l$ relation (c) k_l-k_f relation

Fig. 12 Basic relations of estimating system of permeability coefficient

brass pipes 100 mm in length, 35 mm in inner diameter, and 38 mm in outer diameter and an brass pipe with a length of 110 mm and the same inner and outer diameters for density measurement which are inserted in series into the tester. By carrying out a standard penetration test with a split-spoon sampler with built-in tube, soil samples are caught in the pipes. Thereafter, the split-spoon sampler with built-in tube is pulled up, the pipes in the split barrel are taken out, and the both ends of the pipe second from the front are reformed with an edge knife. Then, the wet density ρ_{tf} of the sample in the second pipe is measured in the field, and laboratory tests of the sample are performed to determine its water content ω_f (%), dry density ρ_{df}, void ratio e_f, and grain-size distribution. It was reported that N-values measured with split-spoon samplers with built-in tube were equivalent to those measured by the standard penetration test (JIS A 1219)[1].

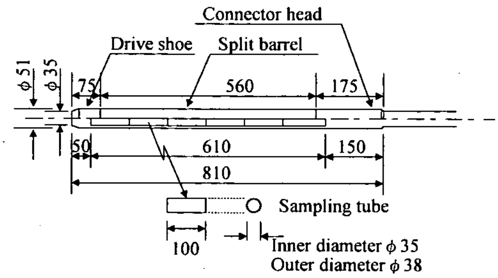


Fig. 13 Split-spoon sampler with built-in tube (In mm)

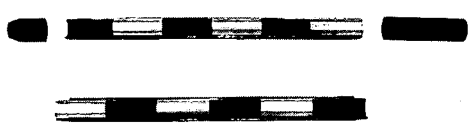


Photo 1 Split-spoon sampler with built-in tube

(2) Basic relations of system at site

a) $N-D_r^*-e_f$ relation

Standard penetration tests were carried out with a split-spoon sampler with built-in tube at six points in the site shown in **Fig. 1**. The maximum grain diameters D_{max} of the samples of the aquifers ranged from 19.0 to 37.5 mm (**Fig. 4**), which were far beyond the limitation 3–5 mm on the maximum grain diameter of the split-spoon sampler with built-in tube[1]. Therefore, the measured void ratios e_f were regarded as different form the actual values due to the crush of grains, etc. at the time of sampling. In addition, because the grain-size distribution of a sample obtained with a split-spoon sampler with built-in tube differs from that in situ, the relative density in situ can not be determined by carrying out maximum- and minimum-density tests. Accordingly, the N-value in situ obtained at the time of the density measurement with the split-spoon sampler with built-in tube was applied to Meyerhof's equation (2) to estimate the relative density D_r^* in situ.

Fig. 14 shows the relation between D_r^* given by the equation (2) and e_f. The $D_r^*-e_f$ relation of the first aquifer showed a fairly good correlation and its primary recurrent equation was as follows.

$$e_f = 0.84 - 0.0058D_r^* \qquad (r = 0.722) \qquad (4)$$

On the other hand, the $D_r^*-e_f$ relation of the second aquifer showed no correlation. Although the cause of no correlation is unknown, it may have something to do with that the second aquifer was a well-compacted sand and gravel layer with a N-value over 50. However, since the coefficient of permeability was necessary for the analysis, adopted as the void ratio e_f of the second aquifer was the mean (= 0.337) of the void ratios of its samples.

b) $e_l - \rho_{dl} - k_l$ relation

To ascertain the relation among the void ratio e_l, the dry density ρ_{dl}, and the permeability coefficient k_l, ρ_{dl} was set to 1.7, 1.9, and 2.1 g/cm^3 and F_c was set to 5%, 10%, and 20% for each value of the dry density to carry out the laboratory permeability tests of the nine cases. Used in the laboratory tests were samples which were adjusted according to the grain-size distribution curves of the first and second aquifers of **Fig. 4** so as to have the maximum grain diameter of 19 mm. **Fig. 17** shows the grain-size distribution of the grading-adjusted samples. When sample soil was density-adjusted and packed into a tester, the water content for compaction was set to the dry side of the optimal value. The reason of the setting was that the permeability coefficient tended to fall saliently on the wet side[8] and hence stable data could not be obtained on that side. The laboratory permeability tests were performed under a constant- or falling-head condition in accordance with the permeability coefficients.

Fig. 15 shows the test results. The $e_l - \rho_d$ relation in **Fig. 15** was given by the following equation[9], the density of soil grains ρ_s assumed to be 2.7 g/cm^3.

$$\rho_{dl} = \frac{\rho_s}{1+e_l} \tag{5}$$

F_c = 7.3% and F_c = 13.6% are the mean of the first aquifer and that of the second aquifer, respectively, and the $\rho_{dl} - k_l$ relation of each aquifer was obtained by interpolation from the same relations determined by the laboratory tests.

c) $k_l - k_f$ relation

The mean of e_f values measured depthwise of each aquifer at each pumping-test point was substituted for e_l in the equation (5) to calculate the mean field dry density $\bar{\rho}_{df}$ of said aquifer. Then, ρ_{dl} was regarded as equal to $\bar{\rho}_{df}$ in the $\rho_{dl} - k_l$ relations corresponding to F_c = 7.3% of the first aquifer and F_c = 13.6% of the second aquifer in **Fig. 15** to find the laboratory permeability coefficients $\bar{k}_l$ of the aquifers. The field coefficients of permeability k_f determined in Section 3 and $\bar{k}_l$ of the aquifers (shown in **Table 1**) are plotted in **Fig. 16** (both the ordinate and the abscissa

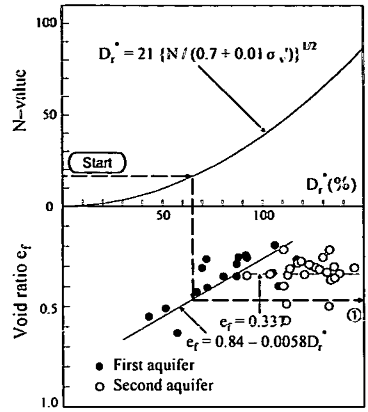


Fig. 14 $N - D_r^* - e_f$ relation

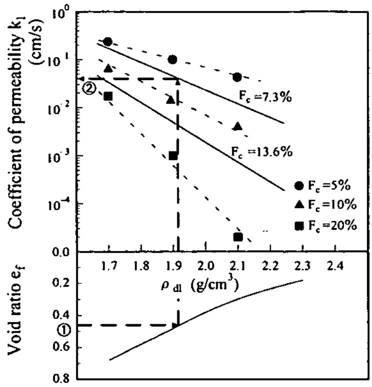


Fig. 15 $e_f(=e_l) - \rho_{dl} - k_l$ relation

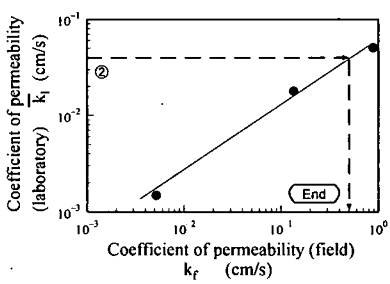


Fig. 16 $k_l - k_f$ relation

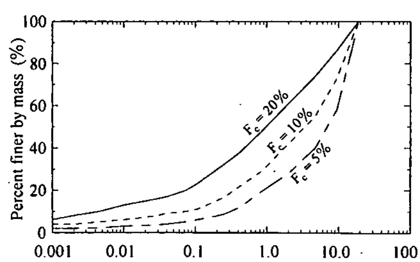


Fig. 17 Grain-size distribution of soil samples for laboratory permeability tests

are in logarithmic scale.).As shown in **Fig. 16**, there is a salient linear relation between k_f and $\bar{k}_l$.

Described in the next chapter is the estimation of the in-situ permeability coefficient k_f from the N-value through the course (Start→①→②→End) shown by broken lines in **Fig. 14–Fig.16**.

5. VERIFICATION OF PROPOSED SYSTEM BY QUASI THREE-DIMENSIONAL SEEPAGE ANALYSIS

The optimal location of the well to draw ground water for the industrial complex and its pump discharge rate of flow had to be determined in order to ensure that the well for the industrial complex would have no significant effects on the existing wells around the complex (the drawdowns in the existing wells should be 5 cm or less). To determine them, the technique of finite-element quasi three-dimensional seepage analysis (GWAP) [10] was used. In such a regional ground water analysis, it is the usual approach that an initial water-level analysis is first performed to verify the analytical model and then the ground water behavior is simulated under given pumping conditions, etc. In the present study, the validity of the proposed system was verified in the initial water-level analyses.

(1) Analytical model and conditions for analysis

Fig. 18 shows a planar finite-element mesh for the area to be analyzed. The boring spots (49 spots) were given planar nodal points, and the intervals between the other nodes were set to about 100–200 m. The vicinity of each pumping-test spot was divided into smaller elements.

As shown in **Fig. 5**, the ground consists, from the surface downward, of the impermeable Ac–ASF layer, the first aquifer, the second aquifer, and the impermeable layers.

The thickness of the aquifers at nodes was determined from the thickness at the boring spots by interpolation with a cubic spline function.

The initial water-level analyses were performed under steady conditions. The analytical model was defined on its west by the boundary between the mountainous area and the level ground, on its east by the Yoshii River, and on its north by a line running east and west at a distance of 700 m from the pumping well, and the water levels at the boundaries of the analytical model were fixed. The reason for setting the north boundary 700 m to the north of the pumping well is that the effects of gravel range 500-1,500 m [11]. The water levels at the

Table 1 Permeability coefficients by pumping tests and laboratory tests

Aquifer	$\bar{k}_l$ (cm/s)	k_f (cm/s)
First aquifer at TB-2	1.7×10^{-2}	1.3×10^{-1}
Second aquifer at TB-2	1.4×10^{-3}	4.9×10^{-3}
First aquifer at BP-6	4.2×10^{-2}	9.8×10^{-1}

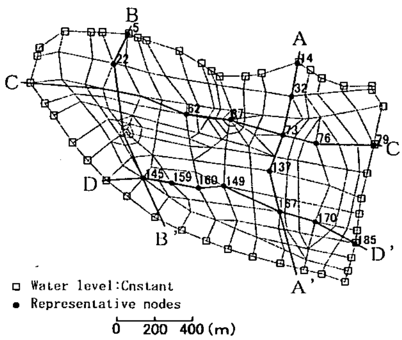


Fig. 18 Division into elements and boundary conditions

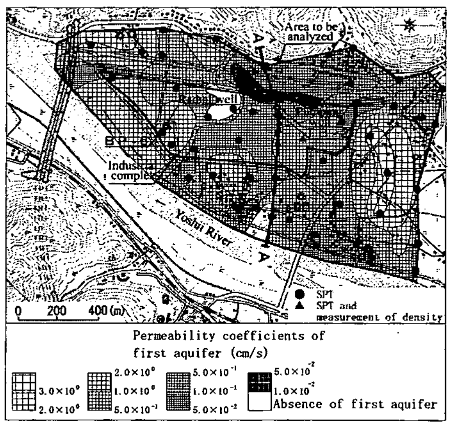


Fig. 19 Distribution of permeability coefficients estimated with proposed system

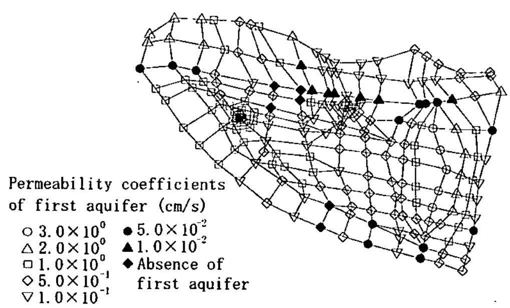


Fig. 20 Permeability coefficients of first aquifer

nodes on the boundaries were measured on May 25, 1992. The permeability coefficients at the nodes in the analysis area were determined as follows. In case of the first aquifer, the N-values measured depthwise at each boring point and F_c (= 7.3%) were applied to **Figs. 14 to Fig.16** to estimate the k_f depthwise. Then, the estimated field mean permeability coefficient $\bar{k}_{f1}$ of the first aquifer at each boring point was calculated. Lastly, the contour map of the $\bar{k}_{f1}$ (**Fig. 19**) was made to determine the permeability coefficients for analysis in accordance with the finite-element division. In case of the second aquifer, the estimated field mean permeability coefficient $\bar{k}_{f2}$ was set to a constant value 5.0 × 10^{-3} cm/s because e_f was assumed to be constant 0.337 as described in the preceding chapter. **Fig. 20** shows the coefficients of the first aquifer estimated with the system.

(2) Results of Analyses and Discussion

Fig. 21 shows the result of quasi three-dimensional seepage analysis performed with the permeability coefficients of the first aquifer shown in **Fig. 20** under steady conditions. As shown in **Fig. 21**, there are fairly large gaps between the measured water levels and the calculated ones in Oseki and Tokutomi areas. It was considered that the gaps had occurred due to the overestimation of inflow from the valley-like hinterland of each area; accordingly, the actual inflows of the areas were estimated with the following equation.

Actual inflow = drainage area × average daily precipitation × infiltration ratio (6)

The drainage areas of Oseki and Tokutomi are 946,900 m^2 and 427,000 m^2, respectively. The average daily precipitation in the drainage areas is 0.0038 m (annual precipitation 1,400 mm/365days). The empirical infiltration ratio is 1/3. **Table 2** shows the actual inflows estimated with equation (6) and the inflows calculated by the quasi three-dimensional seepage analysis.

As shown in **Table 2**, the calculated inflows were more than 10 times the estimated actual ones; therefore, the permeability coefficients of the first aquifer at the nodes 1-36 were reduced on a trial-and-error basis until calculated inflows turned out to be estimated actual inflows. **Fig. 22** shows the node 1-36 and corrected coefficients of permeability. **Table 3** shows calculated inflows after correction.

Fig. 23 and **Fig. 24** show the comparison between the calculated water levels by the initial

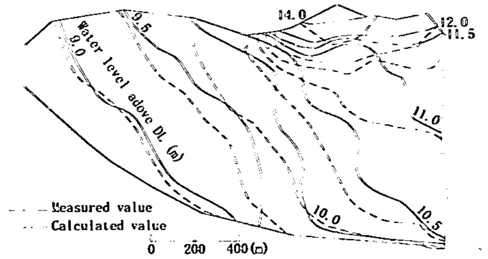


Fig. 21 Result of initial water-level analysis

Table 2 Actual and calculated inflows (m^3/day)

	Estimated actual inflow	Calculated inflow
Oseki area	1,200	14,193
Tokutomi area	540	8,720

Table 3 Actual and calculated inflows after correction of coefficients of permeability (m^3/day)

	Estimated actual inflow	Calculated inflow
Oseki area	1,200	1,777
Tokutomi area	540	557

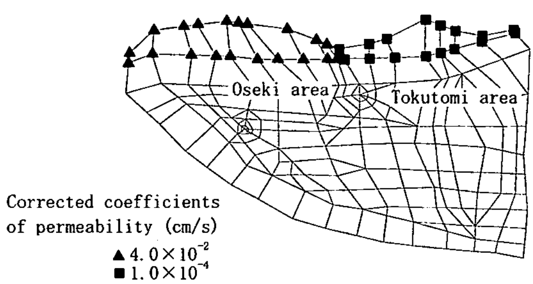


Fig. 22 Corrected coefficients of permeability

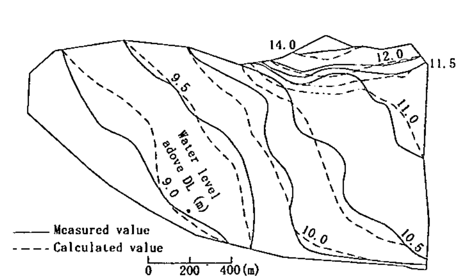


Fig. 23 Result of initial water-level analysis (After correction of permeability coefficients)

water-level analysis after the correction of permeability coefficients and the water levels measured at the 49 boring points. The former and the latter are close to each other. Because the transmissivity T is the most important parameter in the governing equation of the quasi three-dimensional seepage analysis under steady conditions, the excellent agreement between the calculated and measured water levels was suggestive of the appropriateness of the setting of the distribution of T. As can be seen from the equation (1), the accuracy of the proposed system in estimating the permeability coefficient of an aquifer would be fairly high if the thickness of the aquifer at nodes is determined accurately.

6. CONCLUSION

It was suggested that the system for estimating the permeability coefficient proposed by the present study was capable of estimating the spatial distribution of the transmissivity. However, there were the following problems in applying the system to the site of the present study.

a) Meyerhof's equation (2) is applicable only to sand with D_{50} of about 0.2–0.5 mm [12]. Besides, the void ratios e_f obtained by the density measurement with a split-spoon sampler with built-in tube were not the true ones in situ because the sampler was applied to the aquifers of which the maximum grain diameter was far beyond the sampler's limitation. Accordingly, the accuracy of the void ratios e_f obtained by applying N-values to the $N - D_r^* - e_f$ relation was not so good.

b) Because there could be observed no correlation in the $D_r^* - e_f$ relation of the well-compacted second aquifer with the N-value over 50, its e_f was assumed to be a constant value (= 0.337). It meant that the k_f of the second aquifer was fixed to a constant value, and hence the actual behavior of the ground water could not be reflected in the initial water-level analysis. However, since the k_f of the second aquifer was smaller by one order than that of the first aquifer, the effect of the shortcomings on the result of analysis must have been small.

c) Because the $e_l - \rho_{dl} - k_l$ relation of **Fig. 15** was determined by the laboratory tests, there would be no substantial inadequacy in itself. However, k_l obtained by regarding e_f as equal to e_l was not the permeability coefficient corresponding to the true void ratio in situ for the reason mentioned in the above paragraph a). Therefore, reflected in the difference between k_l and k_f shown in **Fig. 16** is difference in density in addition to the commonly recognized anisotropy, grain-size distribution, ground configuration, etc.

d) It is desirable for the permeability coefficients k_f of the aquifers shown in **Fig. 16** and **Table 1**, which were determined by the pumping tests, to be determined by inverse-analysis technique based on seepage analysis wherein the anisotropy of permeability coefficient is taken into account [13), 14)]. The coefficients of permeability k_f were determined by applying Jacob's method to the pumping-test results in the present study; accordingly, this determining method was not sophisticated enough.

Notwithstanding the above problems, the calculated water levels by the initial water-level analyses by using the quasi three-dimensional

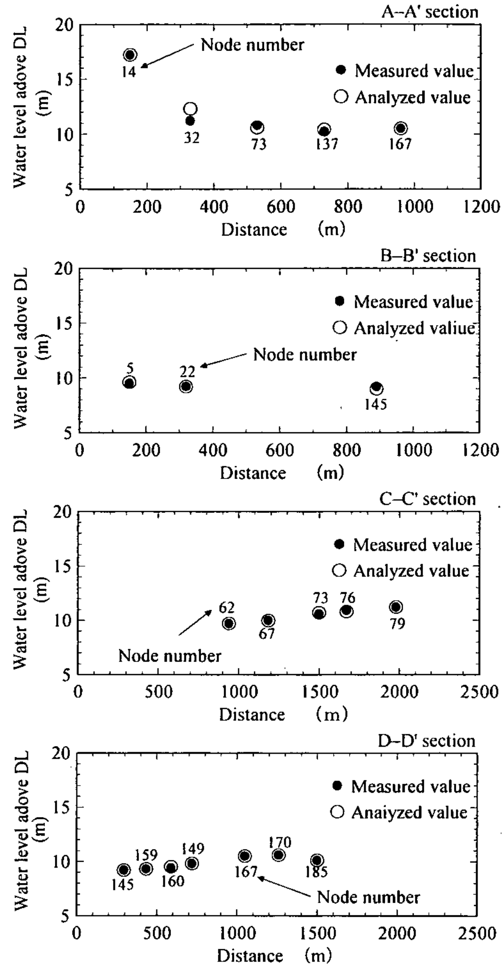


Fig. 24 Result of initial water-level analysis (Sectional view)

seepage analysis method were in an excellent agreement with the measured ones. It may be attributed to the relatively accurate $e_f - D_r^*$ relation which could be obtained because the grain-size distribution of the first aquifer was relatively uniform.

To build the system proposed in the present study, the authors took the position that a technique is useful if it is practical even if a mechanically unknown portion is put in a black box [15]. The system has to be applied to other sites to improve its reliability. At the same time, the problems mentioned above have to be elucidated from a mechanic point of view.

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広域地下水調査における透水係数の推定システム

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準三次元有限要素法などを用いた広域地下水解析を行う場合，透水量係数 T の空間分布の設定が重要となる．その設定方法としては，初期水位解析において実測水位と計算水位が合うように T を任意に変える方法や確率論的に逆解析を用いる方法などがあるが,本報告では，二重管式標準貫入試験器による原位置の密度測定を基にした透水係数 k の推定システムを提案した．また，提案システムの有用性を検討するため，この方法で求めた k を用いた数値解析結果の計算水位と実測水位を比較し，その妥当性を確認した．