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SYNOPSIS: A low test embankment have been constructed on a soft and fibrous peat deposits. The thickness of the soft peat layer is about 2.5 m. Under the peat layer, there is silty clay deposit with thickness of about 13 m. The site investigations were performed for both the layers. The laboratory soil tests on undisturbed samples obtained from both the layers were carried out. The settlement of foundation deposits was predicted by the one dimensional consolidation theory using the soil parameters obtained from laboratory soil tests. The results of field observations were compared with the calculated results by the conventional theory. The results of observations and caluculation do not agree. If the settlement of foundation deposits under embankment loading is predicted by a monitoring method, it appears that a lot of case records of settlement give the conclusion that the agreement of the prediction and the field observation is favourable.

INTRODUCTION

In recent years, peaty ground is often utilized as the foundation of embankment such as road, residential district and others. It is known that peat deposit is highly compressible compared with most mineral soils. The general condition of peaty ground is typically very high in water content and is usually of extremely low bearing capacity. The major involvement of peat in engineering work is in its use a foundation material. In this role, the high compressibility of peat stands out as a most significant engineering property. The consolidation settlement of peat will now be considered as the total compression resulting from volume change under a vertical load.

One of the most striking differences in the compression of peat deposit as compared to mineral soils is the long-term compression which appears to be an almost continuous process. The most part of the continuous settlement have shown a straight-line relation with the logarithm of time [Hanrahan,1964, Molisz et al.,1973].

In applying the conventional theory to the consolidation of peat, there are two major deviations from the usual assumptions, that is, the compressibility of solids and the change in permeability under applied load[Matsuo,1986]. These two anomalies are believed to account for the significant differences in consolidation behaviour between peat and mineral soils. Settlement behaviour of peat under embankment loading has reported[Noto,1987, Lefebvre,1984]. However, the prediction of settlement remain difficult owing to the heterogeneity of peat deposit.

A test embankment was recently bult on peat deposit to study the settlement of low embankment. The site where the test embankment was constructed is located in Saitama prefecture, Japan. The peat layer is about 2.6m thick. There is underlying silty clay layer about 13 m thick. The water level in the peat layer was about 40 cm below the groung surface. The peat and undering silty clay samples were obtained before the construction at the location of the test embankment. Various identification tests and one-dimensional consolidation tests were conducted on the samples.
The fill thickness is 2.0 m. Two test section were builted. In the first test section, the sand mat in thickness of 50 cm was laid between the ground surface and the bottom of the fill. In the second test section, the sand mat was not laid. The wide and length of the fill is 45 m and 120 m respectively. The instrumentation consisted of settlement plates, inclinometers, displacement piles and porewater pressure cells. All instruments were put in place

before and during the costruction. The results of settlement observations were compared with the results calculated by the conventional one-dimensional consolidation theory using the soil parameters obtained from laboratory tests. The field observation give slower settlement than the predicted ones by the laboratory tests for peat layer. However, the field observations give faster settlement than the predicted ones for the silty clay layer.

If the settlement of the peat layer under the low embankment loading is predicted by a monitoring method, it appears that a lot of case records of settlement give the conclusion that the agreement of the prediction and the field observation is favourable.

OUTLINE OF EMBANKMENT CONSTRUCTION

Soil Profile of Foundation Deposits

The low test embankment was constructed in Saitama prefecture. The area consists of the terrace ranging from +12 m to 14 m above sea level and alluvial valley which eroded the terrace. The terrace is diluvial deposit and called "Kanto loam". The Kanto loam is volcanic sedimentary soil. The alluvial deposit is featured with cohesive soil of drowned valley fill and the upper peat soil as shown in Figure 1.

The low test embankment was constructed on the alluvial deposits. From the site investigations, the alluvial deposits were classified into two layers. That is,

(a) upper peat layer
(b) underlying silty clay layer.

In this study, the upper peat layer and the underlying silty clay layer is called Pt layer and Sc layer respectively. The thickness of the alluvial deposits in the embankment area is almost constant with the depth of about 13 m on north-south direction. The thickness of the alluvial deposits on east-west direction varies from 7 to 16 m.

In soil exploration practice drilling and sounding have been made to the embankment area at required interval in both the north-south and the east-west directions.

Figure 2 shows the soil profile and the other soil properties. N and q_c value was obtained by the standard penetration test and the Dutch cone penetration test respectively. The natural water content w_n and the wet density ρ_t was measured from the sample obtained by thin-wall piston sampler. The unconfined compression test was carried out for the undisturbed sample and the unconfined compression strength q_u was also shown in Figure 2.

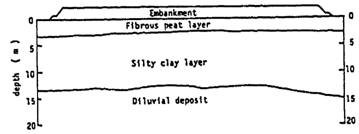

Figure 1 (a) Cross-section of north-south direction of foundation deposits

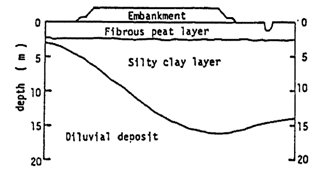

Figure 1 (b) Cross-section of east-west direction of foundation deposits

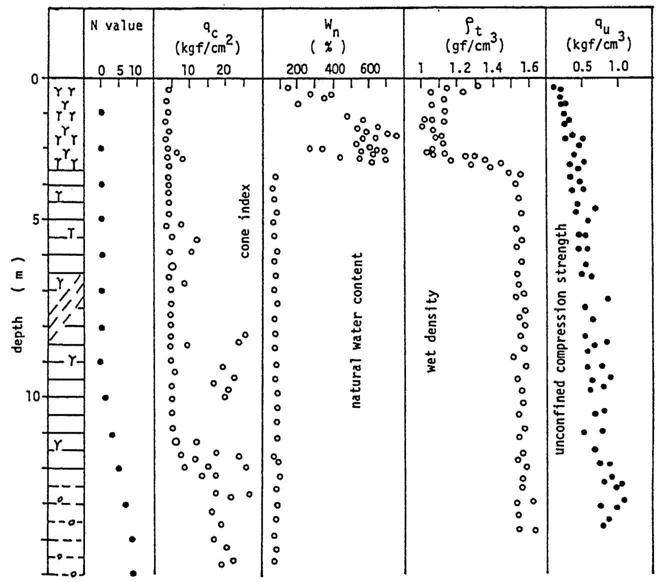

Figure 2 Soil profile and others of foundation deposits

Mechanical Properties of Deposits

The undisturbed samples of peat and silty clay were obtained by the thin-wall piston sampler before the embankment construction. Figure 3 shows the variation of the preconsolidation pressure p_c in relation to depth .

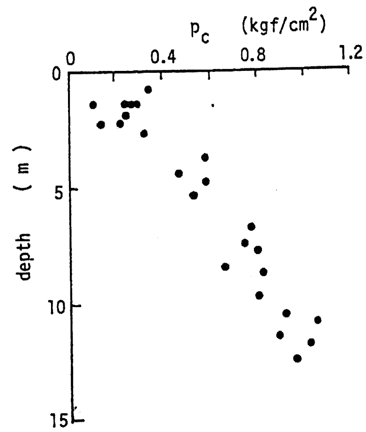

Figure 3 Variation of preconsolidation pressure in relation to depth

It is observed from Figure 3 that the preconsolidation pressure p_c of the peat is in the range between 0.1 and 0.35 kgf/cm^2 and the average value is 0.25 kgf/cm^2. The value is considerably larger than the effective overburden pressure and it is noticed that the peat layer is overconsolidated. The overconsolidation of surface layer in this case is attributed to desication. The preconsolidation pressure p_c of the silty clay is in the range between 0.32 and 1.05 kgf/cm^2. In the silty clay layer, p_c is also considerably larger than the effective overburden pressure.

The relationships between the coefficient of volume compressibility m_v and the consolidation pressure p were shown in Figure 4. It is found that the relationships are almost linear for values beyond the preconsolidation

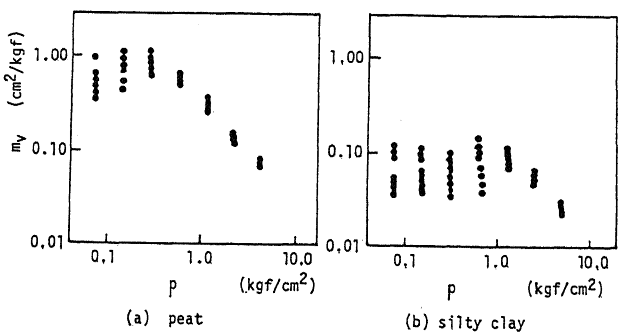

Figure 4 Relationships between volume compressibility m_v and consolidation pressure p

pressure. Figure 5 shows the relationships between the coefficient of consolidation c_v and the consolidation pressure p. The final settlement and the time-settlement relation was calculated by using the results shown in Figures 4 and 5.

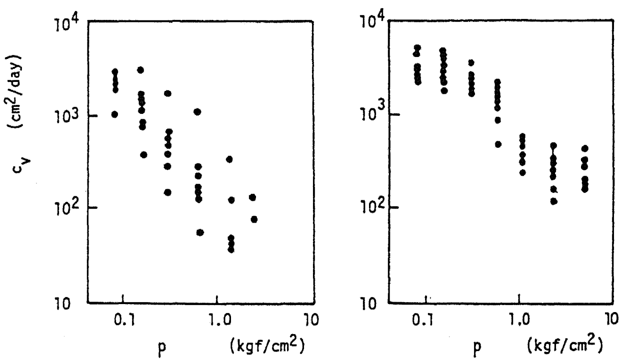

Figure 5 Relationship between coefficient of consolidation c_v and consolidation pressure

Low Test Embankment

The plan of the low test embankment was shown in Figure 6. The gauge arrangement was also shown in Figure 6. The embankment can be divided into two sections. In this study, the section of left hand side in Figure 6 is called section 1. In the section 1, the sand mat with 50 cm thick was laid between the ground surface and the bottom of fill. The section of right hand side in Figure 6 is called section 2 and the sand mat was not laid in the section 2.

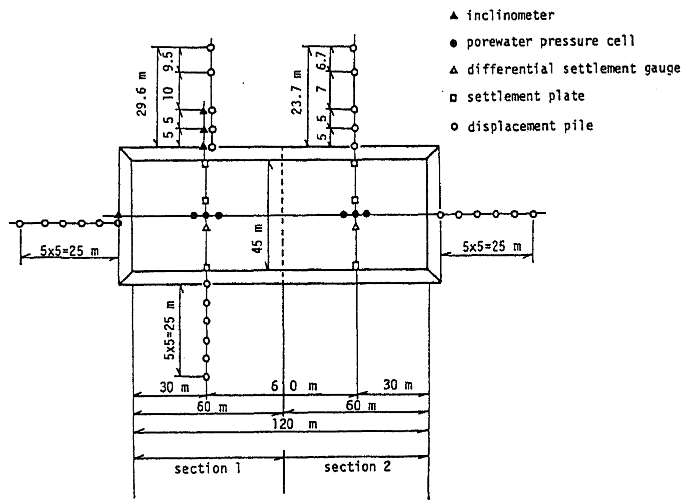

Figure 6 Plane of embankment and arrangement of instruments

FIELD OBSERVATIONS

All the instruments were put in place before and during the construction. The arrangement of instruments was shown in Figure 6. The settlements of the foundation deposits at the center and periphery of the sections 1 and 2 were observed using the settlement plates. Using the differential settlement gauge, the settlements of the peat and underlying silty clay layers were separated.
The settlement observations separated were carried out at the center of the sections 1 and 2. The ground surface movements due to the embankment loading were surveyed using the displacement piles in the area from the toe of embankment to about 25 m outside. The lateral displacements at from the surface to the depth of 20 m in the foundation deposits were measured using the inclinometers in the area from the toe of embankment to 10 m outside. The porewater pressures in the peat and the silty clay were measured by the porewater pressure cells. At the center of sections 1 and 2, the groundwater level were measured by the stand-pipe with strainer.

PREDICTION OF SETTLEMENT

Prediction methods of settlement can be classiffied in two categories. The first is theoretical: Settlement is predicted by theoretical calculation using soil parameters obtained from soil tests and boundary conditions obtatined from soil investigations. The conventional one dimentional consolidation theory, the analysis by finite element method(FEM) and others fall in the category. The second is monitoring method: Settlement is predicted according to the field observations.

For the first, the next conditions have to be satisfied: (1) Accurate soil parameters are obtained, (2) accurate boundary conditions are obtained, (3) theory is reasonably established. For analiysis by FEM, the determination of soil parameters is very difficult. From the reasons mentioned above, the overestimation of FEM have to be examined oneself[Shibata,1983]. The methods in the second category, that is, monitoring methods are usefull to examine the adequate on the initial design and to revise it. From the facts mentioned above, the prediction by the conventional theory is compared with the field observations and the prediction by the monitoring method.

The hyperbolic curve method and others have been proposed as the monitoring method[Krizek et al,1977, Hoshino,1962, Asaoka,1978, Monden,1963]. For the accuracy of these monitoring methods, a interesting results have been reported[Yoshikuni et al,1981]. That is, if the final settlement is predicted using the observations after the degree of consolidation of about 70 and 60 %, the range of the prediction error is in about 10 and 20 % regardless of the monitoring methods used, respectively. The hyperbolic curve method is used in this report.

The hyperbolic curve method is based on the assumption that the rate of settlement is represented by a hyperbolic curve, It is assumed that the relationship between settlement S and time t is schematically represented by a huperbolic curve as shown in Figure 7(a).

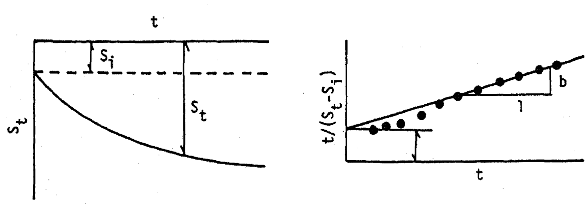

Figure 7 Schematically representation of hyperbolic curve method

The hyperbolic curve is expressed as follow:

$$S_t = S_i + \frac{t}{a + bt} \tag{1}$$

where, S_t is settlement at time t, S_i is initial settlement, a and b are constants obtained from observations. Equation (1) is written as follow:

$$\frac{t}{S_t - S_i} = a + b\,t \tag{2}$$

The relationship between $t/(S_t-S_i)$ and t can be plotted by a straight line as shown in Figure 7(b). The slope of the straight line gives the value of b and the interception of $t/(S_t-S_i)$ axis gives the value of a. The final settlement S_f is given as the value of S_t when $t \to \infty$ and is written as follow:

$$S_f = S_i + \frac{1}{b} \tag{3}$$

RESULTS OF OBSERVATIONS AND DISCUSSIONS

Stability of Foundation Deposits

To examine the stability of foundation deposits during construction, the diagram for construction control of embankment have been proposed[Matsuo et al,1978]. It is very convenient that the stability of foundation deposits is examined using this diagram. It was confirmed that the foundation deposits under the low embankment loading was stable to failure.

Final Settlements

The results of settlement observations were shown in Figures 8 (a) and 9(a). The relationships between t and $t/(S_t-S_i)$ are plotted in Figures 10 and 11.

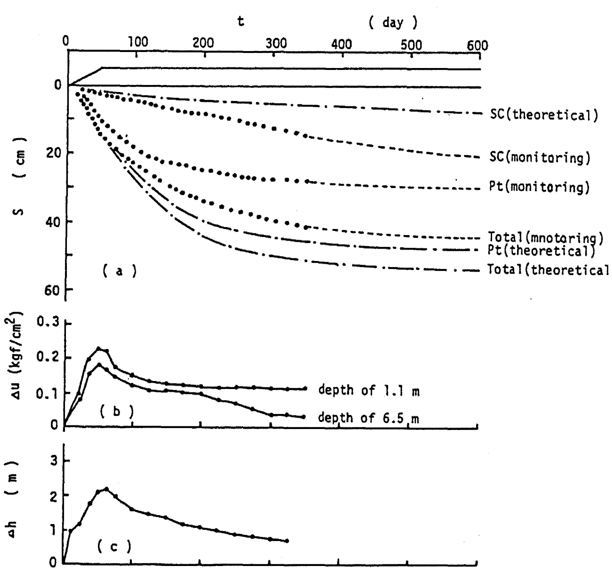

Figure 8 Settlement S, porewater pressure Δu and groundwater level Δh at center in section 1

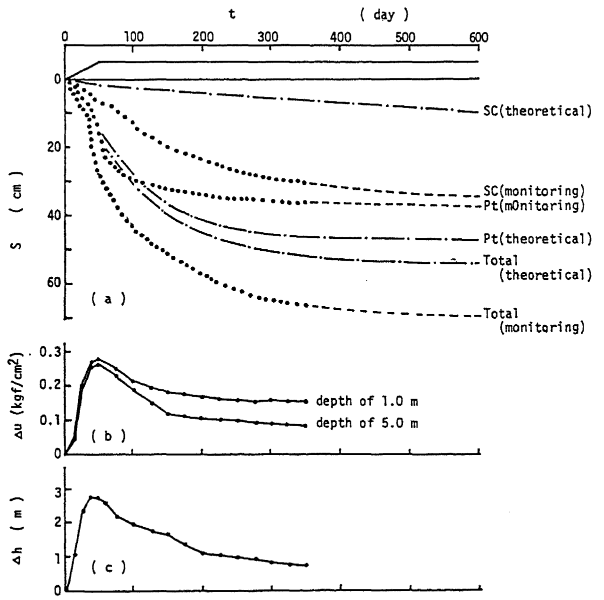

Figure 9 Settlement S, porewater pressure Δu and groundwater level Δh at center in section 2

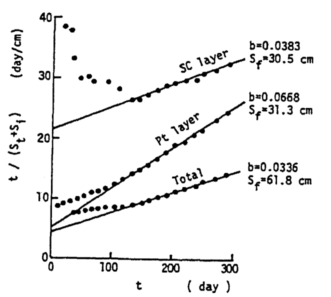

Figure 10 Prediction of settlement by hyperbolic curve method(section 1)

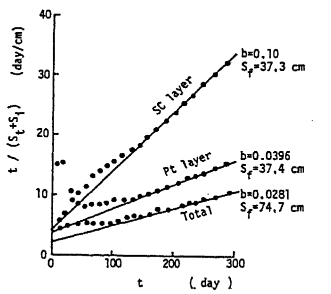

Figure 11 Prediction of settlement of hyperbolic curve method(section 2)

The regression lines can be obtained from the plots shown in Figures 10 and 11. The constants a and b can be obtained from the regression line and are shown in Figures 10 and 11. The final settlement S_f can be calculated from equation (3) and the values of S_f calculated are also shown in Figures 10 and 11.

The final settlements of the sections 1 and 2 calculated by the theoretical method(m_v method) are shown in Table 1 with the results by the hyperbolic curve method.

Table 1 Comparison of final settlement

		final settlement S_f(cm)		
		theoretical (m_v method) (1)	monitoring (hyperbolic) (2)	(2)/(1)
section 1	peat	46.1	31.3	0.68
	silty clay	21.5	30.5	1.41
	total	67.6	61.8	0.91
section 2	peat	46.8	37.3	0.80
	silty clay	24.2	37.4	1.56
	total	71.0	74.7	1.05

For Pt layers, the final settlements by theoretical method indicate the results larger than these by the hyperbolic curve method. On the other hand, for the SC layers, the final settlements by the hyperbolic curve method are larger than these by the theoretical. The prediction of final settlement by the theoretical was found to be far from satisfactory. Better prediction can be expected if more accurate values of m_v and boundary condition are employed.

Time-Settlement Relations

To simplify the calculation of rate of consolidation for the multi-layered foundation, the two strata were converted to a single stratum by the method previously proposed[Palmer et al,1957]. Figures 8(a) and 9(a) show the comparisons between the observed and predicted settlements. For the SC layers, the observed settlements are far larger than predicted ones. On the other hand, for the Pt layers, the predicted settlements are larger than observed ones.

The degree of consolidation U_{350} at 350 days after the beginning of construction is calculated and shown in Table 2. In Table 2, S_{350} is the settlement at 350 days after the beginning of construction. For Pt layers, U_{350} is more than about 90 %. However, the degree of consolidation of SC layers is lower than these of the Pt layers. There is a considerable difference between the degrees of consolidation by the theoretical and hyperbolic methods.

Table 2 Comparison of degree of consolidation U_{350} at 350 days after beginning of constraction

		theoretical			hyperbolic		
		S_f	S_{350}	U_{350}	S_f	S^*_{350}	U_{350}
section 1	peat	46.1	45.2	98 %	31.3	28	89 %
	silty clay	21.5	5.6	26 %	30.5	14	46 %
	total	67.6	50.8	75 %	61.8	42	68 %
section 2	peat	46.8	46.2	99 %	37.3	36	97 %
	silty clay	24.2	6.4	26 %	37.4	31	83 %
	total	71.0	52.6	74 %	74.7	66	88 %

* measured value

Porewater Pressures and Groundwater Levels

The variation of the porewater pressure Δu in the Pt and SC layers uneder the center of the sections 1 and 2 are shown in Figures 8(a) and 9(b). It is seen that the maximum porewater pressure occurs at the end of construction and the values are from 0.19 to 0.29 kgf/cm^2. On the other hand, the increment load by the embankment is 0.35 kgf/cm^2 The maximum porewater pressure occured in foundation deposits are smaller than the increment load. For both the sections 1 and 2, the dispersion of porewater pressure in SC layer is faster than it in Pt layer. The maximum porewater pressure in the section 1 is smaller than it in the section 2 due to the effect of the sand mat.

The groundwater level in the stand pipe at the center of the sections 1 and 2 were shown in Figures 8(c) and 9(c). The maximum groundwater level in the section 1 is lower than it in the section 2. The groundwater levels approximately correspond to the porewater pressures. The degrees of consolidation U_{350} at 350 days calculated from the porewater pressure measured were shown in Table 3.

Table 3 Degree of consolidation obtained from porepressure at 350 days after beginning of constraction

		depth	porewater pressure at 350 days Δu(kgf/cm^2)	maximum porewater pressure Δu_m(kgf/cm^2)	theoretical porewater pressure Δu_o(kgf/cm^2)	degree of consolidation U_{350}(%)
section 1	peat silty clay silty clay	1.2 6.5 (8.0)	0.086 0.070 (0.040)	0.214 0.19 (0.19)	0.37	77 81 (89)
section 2	peat silty clay silty clay	1.0 5.0 (8.0)	0.141 0.104 (0.100)	0.286 0.277 (0.270)	0.37	62 72 (73)

$U_{350}=1-\Delta u/\Delta u_o$

Movements of Foundation Deposits

Figures 12 and 13 show the horizontal movements measured by the inclinometer located at the foot of the slope edge

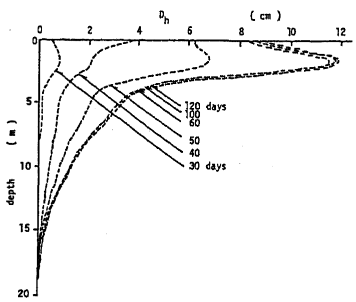

Figure 12 Horizontal Displacement D_h at toe of embankment (section 1)

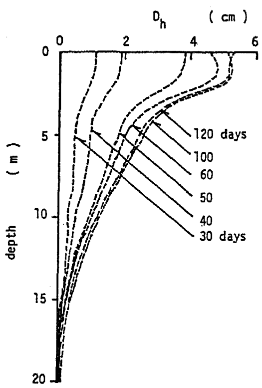

Figure 13 Horizontal displacement D_h at 5 m away from toe of embankment (section 1)

and 5 m away from it in the section 1 respectively. At the end of the measurements, the maximum displacement measured was about 12 cm at the toe of slope and about 5 cm at 5 m away from it. It is noted that the horizontal displacement is considerably small.

Figure 14 shows the vertical movements measured by the displacement piles located at from the foot of the slope edge to 23 m away it in the section 1. At the toe of the slope, a considerable settlement occurred, but the vertical displacement do not nearly occurred at from 5 m to 20 m away it.

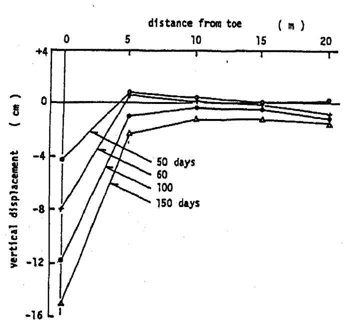

Figure 14 Vertical displacement at from toe of embankment to 20 m (section 1)

SUMMARY

In this report, the measured results of the settlement and others of the soft foundation deposits due to the low embankment were shown. The observations are presently continuing. Therefore, the final conclusion can not be obtained. From the observations until the present, the followings can be summarized.

The final settlement by theoretical method do not agrees with it by the monitoring method. The rate of settlement by the conventional theory do not also agrees with the field observations. The theoretical prediction must be used as the probable value at the initial design and the prediction during and after construction must be corrected by the monitoring method.

We would like to express our thanks to the Japan Hosing Corporation for having made available their observed data for inclusion in this report.

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