5.2 Result of Soft Ground Analysis

For a representative case of soft ground along the railway, analysis of soft ground by the Ground Surface Treatment method (Use of Sand Mat) and Replacement method (depth 0.5m) is given below.

5.2.1 Soft Ground and Embankment Condition

(1) Condition of Soft Ground

1) Extent of soft ground, and ground water level: refer to table of soil section and design soil value

2) Design parameters of soft ground

$\gamma t = 1.700\ tf/m^3$

$qu = 3.00 \sim 6.00\ tf/m^2$

$Co = 1.50 \sim 3.00\ tf/m^2$

$Py = 6.00 \sim 12.00\ tf/m^2$

Ratio of strength to consolidation pressure m = 0.25

Figure 3-3-1 e-logP Design Curve

Figure 3-3-2 log Cv-logP Design Curve

(2) Embankment Details

Embankment detail refer section 5.1.2

5.2.2 Settlements and Stability Analysis for Embankment

High embankments in this project area are shown as below

Km 0+750～1+100 Representative km 1+050 Embankment height is 10.0 m and 5.0m

Km 2+150～2+200 Representative km 2+150 Embankment height is 5.0 m

Km 17+900～18+000 Representative km 17+900 Embankment height is 3.5 m

Km 22+800～23+100 Representative km 23+100 Embankment height is 5.7m

The settlement and stability analysis for embankment was carried out the above-mentioned representative section.

The results of analysis are shown as Table 5.2.1 (a) and (b)

Table 5.2.1 (a) Results of analysis

Representative Km Age	Embankment Height HE(m)	Logical Settlement	Time (U=80%)	Safety Factor Fs
1+050	10.0	90.5 cm	14.8 years	0.835
	5.0	48.3 cm	14.2 years	1.634
2+150	5.0	37.7 cm	1.9 year	1.654
17+900	3.50	16.7 cm	1.2 year	2.129
23+100	5.70	37.5 cm	3.5 years	1.355

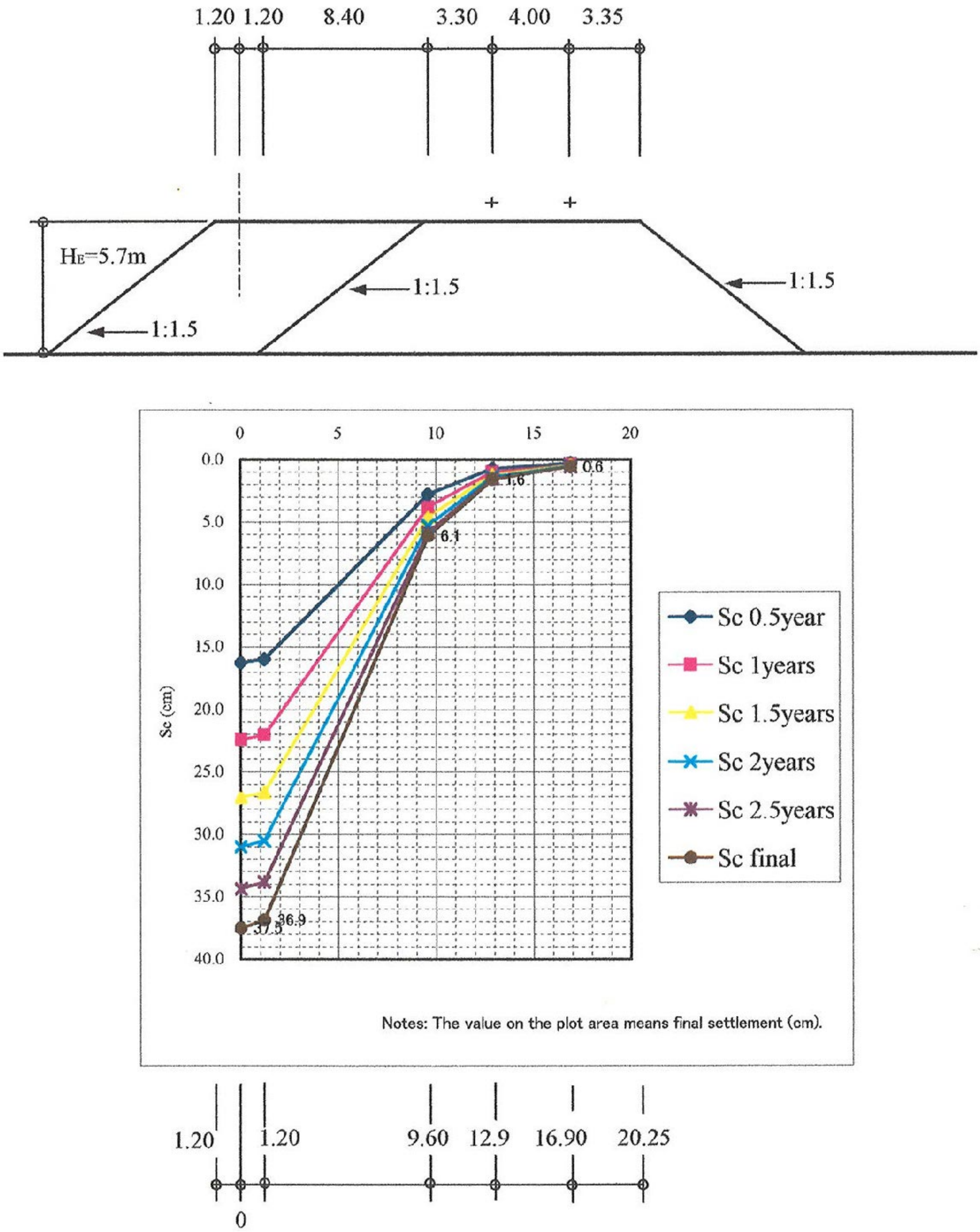
From these results, the safety factor at Km 1+050 HE=10m is 0.835, which is less than 1 and consequently construction of an embankment HE=10m is impossible.
Consequently, the embankment height is reduced HE=5.0 meters. The results in the table represent the duration of settlement and the maximum value on the centre line of the embankment.
These embankments are expected to influence existing or new railway. Its check results are shown Table 5.2.1 (b) influence to existing or new railway and Figure 5.2.1 to Figure 5.2.4 relationship of settlement and distance.

Table 5.2.1 (b) Influence upon existing or new railway

Km Age	Embankment height HE (m)	Distance from toe of slope	Settlement Value of Railway
2+150	5.00	13.2 m	0.5 cm
17+900	3.50	3.30 m	0.23 cm
		7.30 m	0.08 cm
23+100	5.70	3.30 m	1.6 cm
		7.30 m	0.6 cm

Reference: In accordance with Japan Building Standard, Limited of Influence to Building by Differential Settlement is 1/250 under.

Km.23+100 HE=5.7M

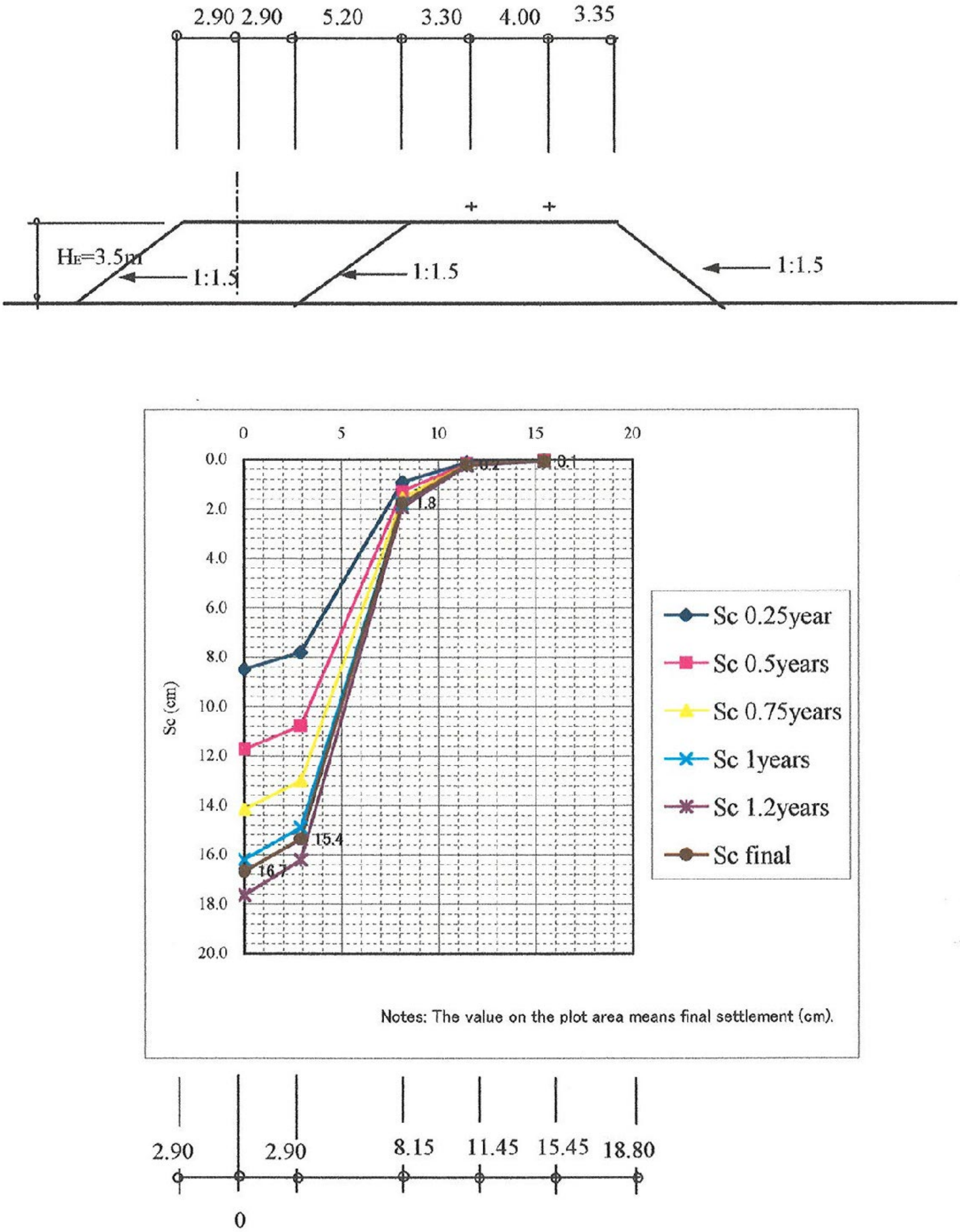


Final settlement (cm)

Distance (m)	0	1.20	9..60	12.90	16.90
Settlement (cm)	37.5	36.9	6.1	1.6	0.6

Fig. **Settlement and Distance Km.23+100 H=5.7m**

Km.17+900 HE=3.5m

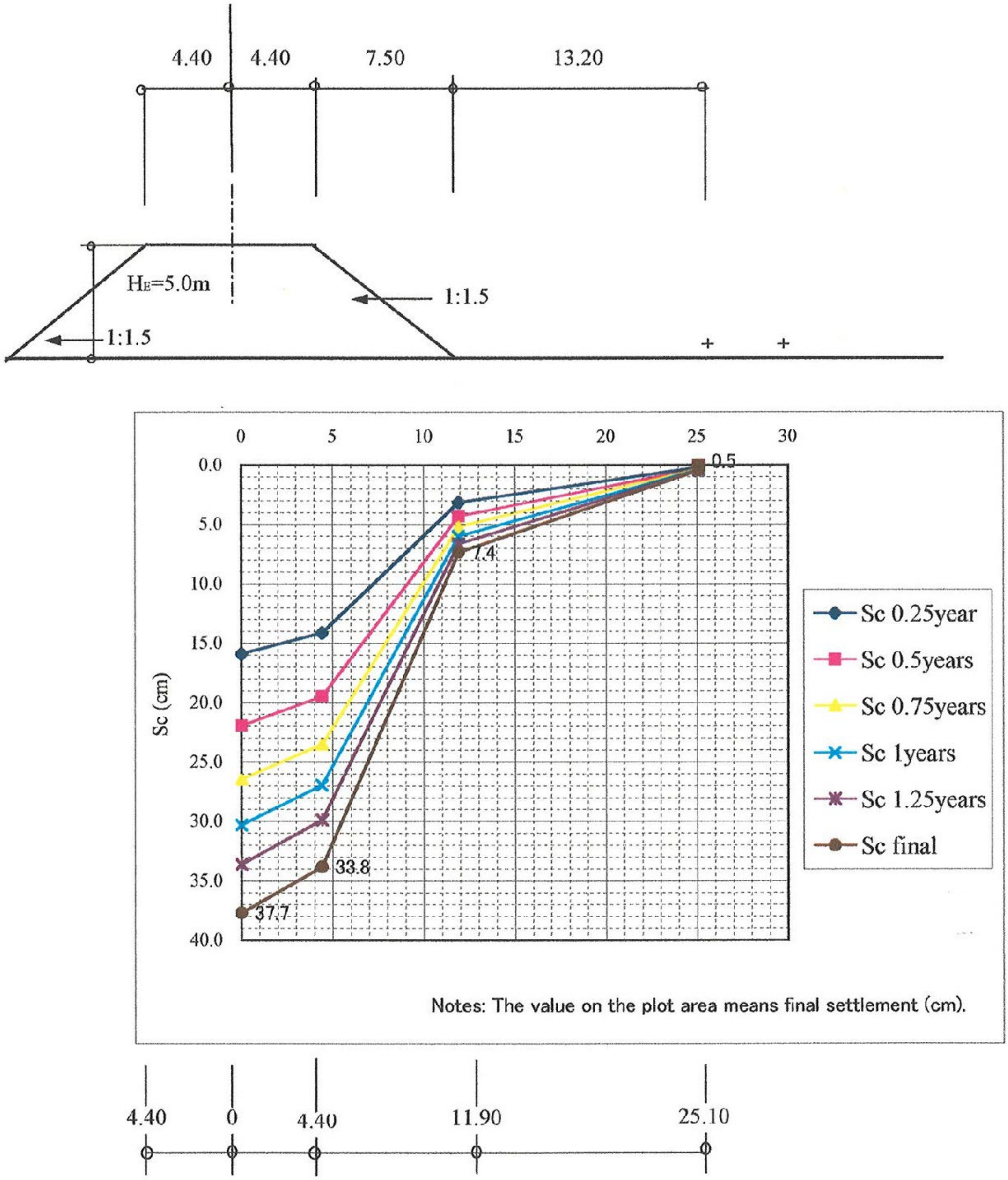


Final settlement (cm)

Distance (m)	0	2.90	8..15	11.45	15.45
Settlement (cm)	16.7	15.4	1.8	0.2	0.1

Fig. 5-2-2 **Settlement and Distance Km.17+900 HE=3.5m**

Km.2+150 HE=5.0M

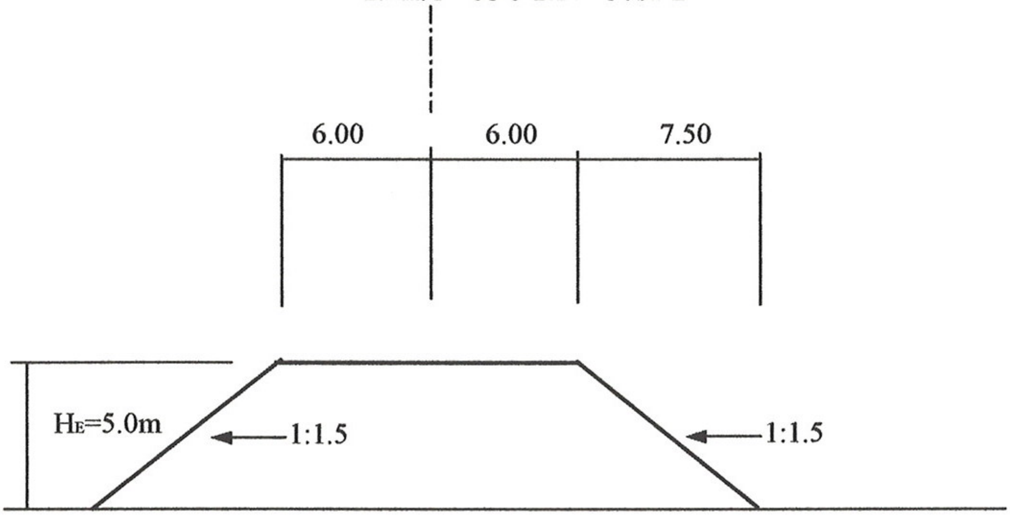


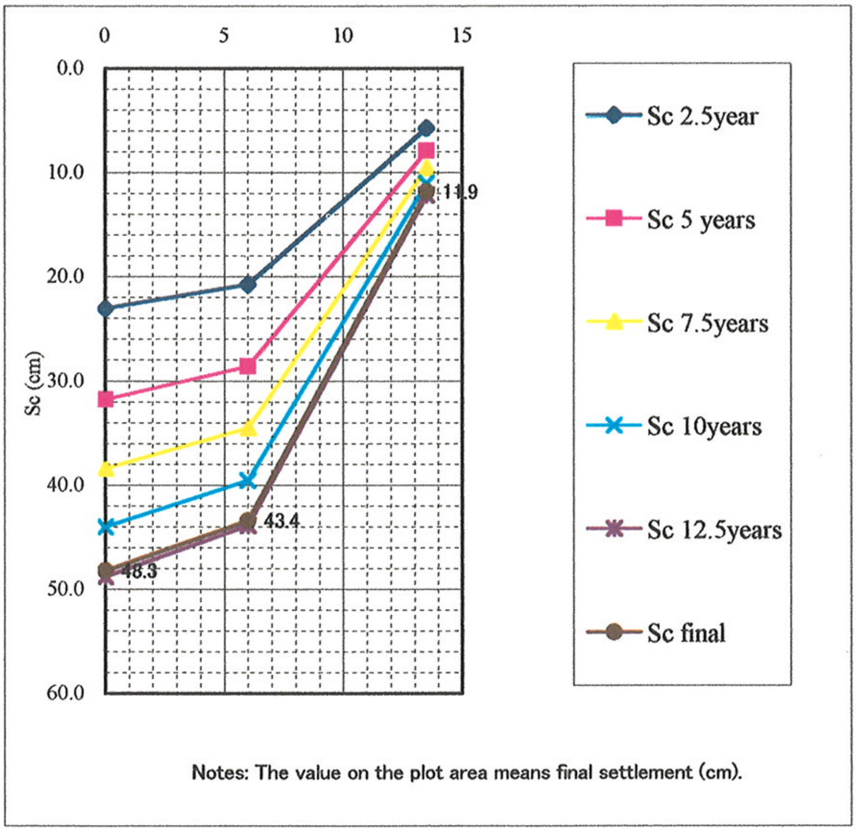
Final settlement (cm)

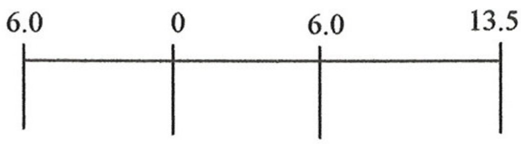
Distance (m)	0	4.40	11.90	25.10
Settlement (cm)	37.7	33.8	7.4	0.5

Fig. 5-2-3 **Settlement and Distance Km.2+150 HE=5.0m**

Km.1+050 HE=5.0M







Final settlement (cm)

Distance (m)	0	6.0	13.5
Settlement (cm)	48.3	43.4	11.9

Fig. 5-2-4 **Settlement and Distance Km.1+050 HE=5.0m**

5.3. Foundation for Structural Design

5.3.1. Coefficient of the Soil Reaction

(1) Objective

The purpose of the pressure meter test is to measure lateral distortion characteristics of the ground. Basically, lateral pressure-meter test in a borehole is carried out in-situ, which are planned the previous of the structure. However, in the investigation, the test was not carried out. Therefore, the problem is obtained by an indirect method.

(2) Calculated from N-Value and E-Value.

K-Value is calculated from E-Value, which it is obtained from standard penetration test N-Value (sandy soil) and from unconfined compressive strength test E_{50}-Value (cohesive soil)

(3) Relationship of E-Value and N-Value

Relationship of the lateral pressure-meter test in borehole E-Value and standard penetration test N-Value has been studied by many analysts. Those results, in all kinds of soil, E=700 N kN/m^2 (E=7N kgf/cm^2) are formed approximately.

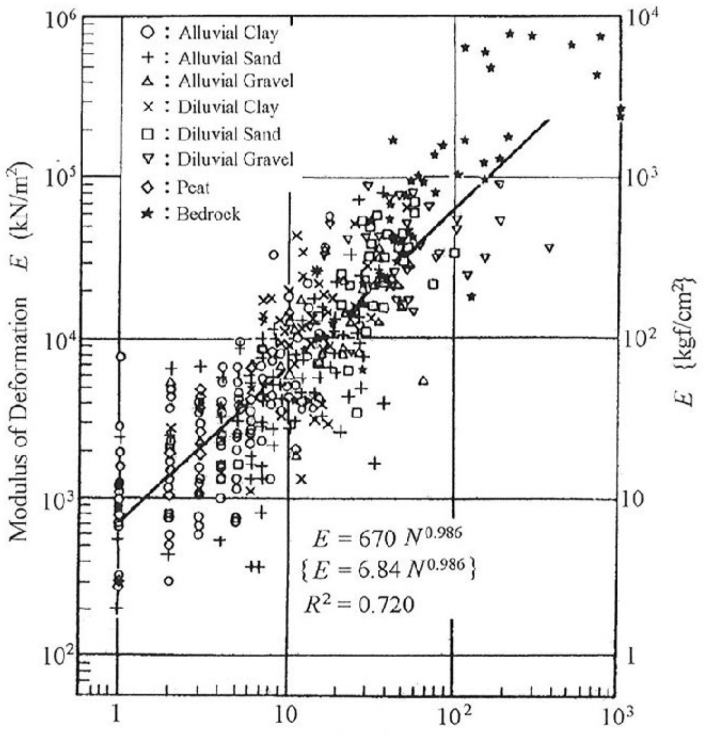


Figure 5.3.1 Relativity of N-Value and Value of modulus of deformation

(4) Relationship of E-Value and Laboratory soil test

In order to estimate the modulus of deformation obtained from lateral loading test in a borehole. Many analyses on the relationship between the modulus of deformation obtained from lateral loading test in borehole and from laboratory soil test such as the unconfined and/or triaxial compression tests have been carried out. Following figure 4.4.2 is an example. Both modulus of deformation have agreement in all types of cohesive soil.

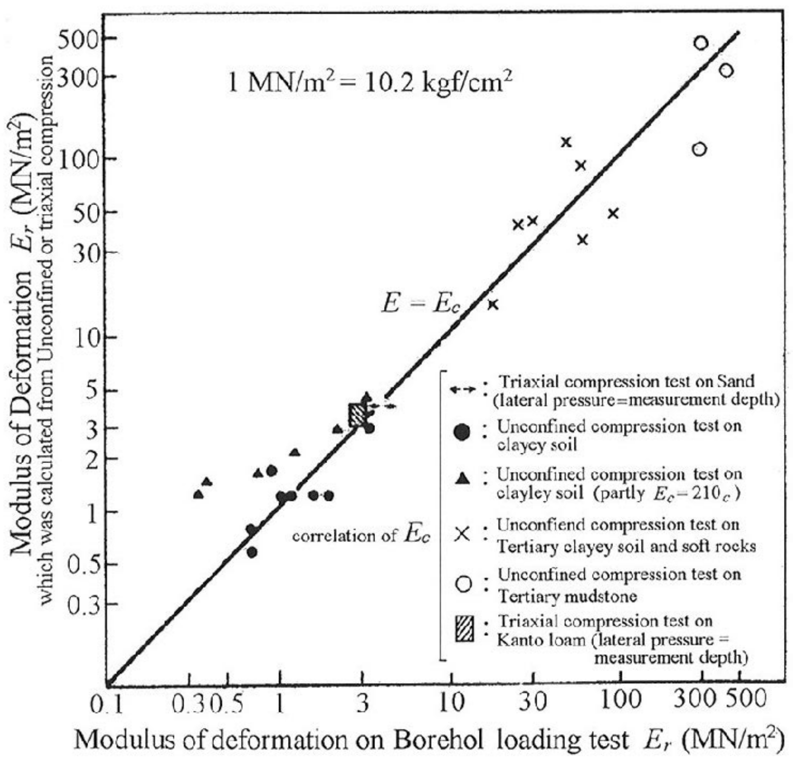


Figure 5.3.2 **Relativity of modulus of deformation, obtained from Lateral loading test in borehole and Laboratory soil test**

In this investigation, it's obtained from:

$$E_{50} = 22.925qu + 1.253$$

Reference: Study of mechanical property around Osaka in Japan (Tuneo Imai 1972)

$$E = 26.5qu$$

(5) In-situ Test (Lateral Load Test in Borehole)

Lateral load test in borehole is commonly used in Japan as the Pressure meter Testing using OYO LLT (LLT-M25 type) devices. These tests were carried out for this project and the test results are shown table 5-3-2.

(6) Coefficient of lateral ground reaction

In the Japanese Road Bridge specification and manuals and the Pile foundation design handbook, coefficient of horizontal ground reaction kH (kN/m^3) {kgf/cm^3} is calculated using the modulus of deformation E (kN/m^2){kgf/cm^2} which is obtained from horizontal loading test in borehole.

$$k_H = \frac{1}{0.3} \alpha_H E \left(\frac{B_H}{0.3}\right)^{-3/4}$$

$$\left\{ k_H = \frac{1}{30} \alpha_H E \left(\frac{B_H}{30}\right)^{-3/4} \right\}$$

Where :

α_H: Factor, which is estimated ground reaction factor is always advised as 4 for the Borehole loading test, and for earthquakes is 8.

B_H: Converted loading width (cm) of foundation, which crosses at right angles with the load working direction as shown Table 5.3.1

A_H: Loading area of foundation which crosses at right angles with load working direction (cm^2)

D: Loading width of foundation which crosses at right angles with load working direction {cm}

β: Characteristic vale of foundation as following expression

$$\beta = \sqrt[4]{\frac{k_H D}{\sqrt{4 E_F I}}}$$

$E_F I$: flexural rigidity of foundation (kN/m^2) {kgf/cm^2}

Table 5.3.1 Converted loading width of foundation

(Japan Road Bridge Specification and manual)

Foundation Style	B_H(m) {cm}
Spread foundation	$\sqrt{A_H}$
Caisson foundation ($\beta l < 1$)	$\sqrt{A_H}$
Caisson foundation ($1< \beta l < 2$)	$\sqrt{D / \beta}$
Pile foundation	$\sqrt{D / \beta}$
Steel pipe Steel pile foundation	$\sqrt{D / \beta}$

Source: Handbook of site investigation for geotechnical engineering.

(The Japanese Geotechnical Society)

Table 5-3-2 SUMMARY OF LATERAL LOAD TESTS

BH-NO.	Depth of LLT (m)	Earth Press. at Rest. Po (Kg/cm^2)	Yield Press. Py (Kg/cm^2)	Failures Press. Pf (Kg/cm^2)	Coef. of Soil Reaction. Km (Kg/cm^3)	Modulus of Elasticity Em (Kg/cm^2)	Mean Radius of k Value rm (cm)	Notes
5	2.0	0.70	2.00	2.75	2.36	13.57	4.08	
	4.0	0.50	4.50	5.75	11.11	55.85	3.78	
	6.0	0.45	1.70	2.56	5.21	25.05	3.67	
	8.0	0.90	7.70	9.40	45.33	259.87	3.90	
6	2.0	0.50	4.60	6.20	24.12	114.84	3.58	
	4.0	0.70	5.40	6.50	29.37	154.05	3.72	
	6.0	0.90	4.30	5.25	9.71	56.38	3.95	
3	2.0	0.90	2.20	3.00	21.66	116.85	3.67	
	4.0	0.71	3.75	4.50	17.88	94.29	3.74	
	6.0	0.80	2.50	3.25	9.44	54.24	3.99	
	8.0	0.62	2.35	3.35	8.77	45.50	3.76	
14	2.0	0.60	2.30	3.30	7.39	40.29	3.98	
	4.0	1.00	2.81	4.60	1.80	98.82	3.66	
	6.0	0.50	3.65	4.32	10.50	52.78	3.78	
	8.0	0.80	3.50	5.20	3.00	158.98	3.68	
15	2.0	0.57	1.97	3.06	8.23	40.40	3.61	
	4.0	0.75	3.60	4.40	9.19	49.54	3.77	
	6.0	0.80	2.50	4.70	3.40	19.58	4.00	
	8.0	0.46	1.42	1.75	5.05	23.95	3.62	
16	2.0	0.65	1.75	2.25	5.00	25.51	3.67	
	4.0	0.95	3.00	4.16	14.64	78.22	3.61	
	6.0	0.45	2.60	3.70	16.54	77.57	3.58	
	8.0	0.45	1.40	1.87	3.80	19.06	3.83	
4	2.0	0.48	0.55	0.81	3.50	17.23	3.73	
	4.0	0.48	1.18	1.36	5.00	24.53	3.69	
	6.0	0.45	1.45	1.72	4.76	23.38	3.75	
18	2.0	0.46	0.92	2.18	1.21	6.51	4.11	
	4.0	0.46	1.82	2.50	8.50	40.09	3.60	
	6.0	0.80	3.60	4.40	9.03	49.15	3.78	
	8.0	0.57	2.60	3.60	7.22	36.13	3.68	
29	5.0	0.87	2.13	4.96	11.79	65.53	4.28	
30	4.4	0.94	3.06	5.00	12.48	75.72	4.67	
31	5.0	0.94	2.24	3.22	11.48	60.78	4.07	
n		33	33	33	33	33	33	
Max.		1.00	7.70	9.40	45.33	259.87	4.67	
Min.		0.45	0.55	0.81	1.21	6.51	3.58	
Average		0.670	2.759	3.805	10.560	62.856	3.818	
σn		0.187	1.436	1.741	9.035	52.118	0.233	
Average-σn		0.483	1.324	2.065	1.525	10.738	3.585	
Average+σn		0.857	4.195	5.546	19.595	114.973	4.051	