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Excessive Settlement Back-Analysis of Railway Embankment on Soft Soils during Service

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ABSTRACT : This paper presents case history of railway embankment excess settlement on soft clay during service in southern region of Korea. A lot of field observations show that the measured settlements are a lot larger than settlements actually calculated in this area. Back analysis is carried out to verify the soil parameters which are intended to investigate in the subsurface exploration phase and later in a laboratory test program. Recommendations and causes for the engineering practice are suggested to review the determination of excess settlements and, consequently, to improve the settlement prediction. This enormous discrepancy is due to the passing over secondary consolidation, and the design filling did not meet to real construction filling. Immediate settlement could be subsidiary factor of excess settlement.

Keywords : Back analysis, Soft clay, Consolidation, Excess settlement

1. Introduction

This paper demonstrates several observation results of soil consolidation under railways on soft clays in southern part of Korea. The railways started service approximately five years ago, but the settlement has not ended contrary to designer's expectation. The observations were performed in the station and its environs. The measurements were carried out far enough to form a reliable basis to study the primary settlement and the secondary settlement to some extent. The main purpose of this paper is to demonstrate the considerable discrepancy between the measured and the calculated settlement as well as to explain it using the back-analysis associated with a laboratory test program, where standard consolidation test and constant rate of loading consolidation test are carefully carried out. The influences of the load increment and the loading rate on the soil deformation behavior have been discussed. Finally recommendations for the engineering practice are suggested that may lead to better design parameters for settlement estimation in this type of soils.

2. Soils

2.1 Soil layers

The soil profile near soft ground site is shown in following Table 1. It shows very shallow depth of groundwater and 20～30 m thickness of clay layer. This thickness of clay is

Table 1. Soil profiles

Sections	Borehole	GL (m)	GW (m)	Clay thickness (m)
1	BB--8	107.58	0.7	23.5
	BB--9	106.71	1.2	21.7
	BB--10	106.30	1.6	22.1
	BB--11	106.08	1.4	25.0
	BB--12	106.37	2.7	20.7
	BB--13	105.57	0.6	25.3
2	BB--14	106.48	1.5	28.4
	BB--15	106.60	1.4	21.5
	BB--16	106.07	3.1	28.4
	BB--17	106.24	3.5	27.7
	BB--18	106.01	1.3	27.8
	BB--19	106.09	3.6	23.7
3	BB--20	106.15	1.1	25.5
	BB--21	106.65	0.9	23.7
	BB--23	106.57	1.6	25.0
	BB--24	106.74	3	24.5
	BB--26	106.60	0.0	23.4

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considered as not too big as compared with several cases in southern part of Korea. Soil of Section 4 is little bit thicker than other sections. Soil of BB-15 shows smaller value of thickness than adjacent holes, but still dark gray silty sand of BB-15 may be regarded as silt. BB-15 soil could show same thickness if this soil is included in compressible soils.

2.2 Physical properties of soils

In geotechnical terms, those sensitive subsoils are silt clays (CL) to high plastic clay (CH) with soft to very soft consistency, while water content is up to 30~60%. Because of high compressibility the clays in southern Korea are considered as difficult soil in foundation engineering and loads of normal structures are typically carried by deep foundations. For the purpose of classification, grain size analysis and Atterberg limits tests of the soft clay from several sites were carried out. The results with additional data are presented in Fig. 1 and Fig. 2. The passing percent of No. 200 sieve is over 90%, and then the soil can be regarded as typical clay. The certain amount of silty soil is located at upper part of soil layers, so it is considered that compression index is small and consolidation velocity is fairly large.

Liquid index (LI) is average 1.0 except a few high values could be regarded error values. This means that soil is very plastic, and has very low strength.

In addition, results of physical and mechanical properties such as undrained shear strength measured from three locations are shown in Table 2. The strength increase ratio, defined as undrained shear strength divided by effective overburden

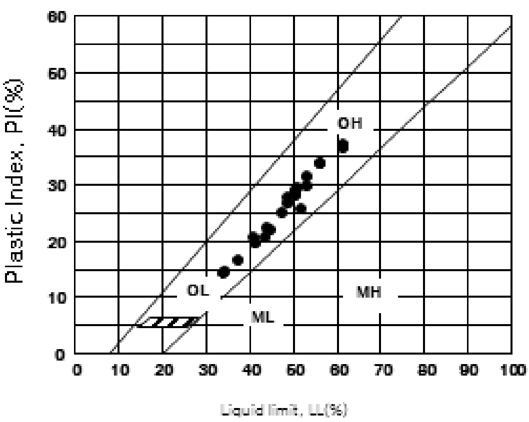

Fig. 1. Plasticity chart of clay soils

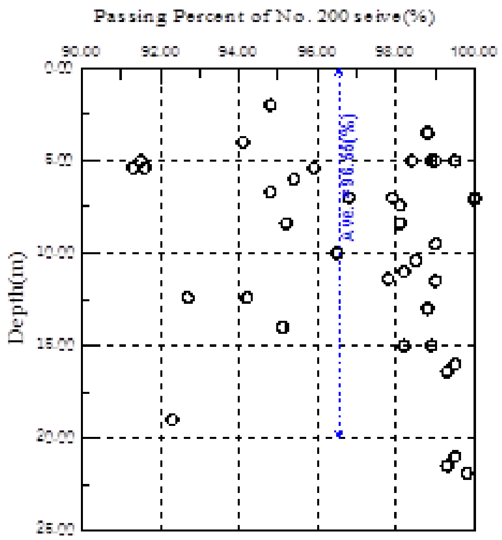

Fig. 2. Distribution of passing percent of No.200 sieve

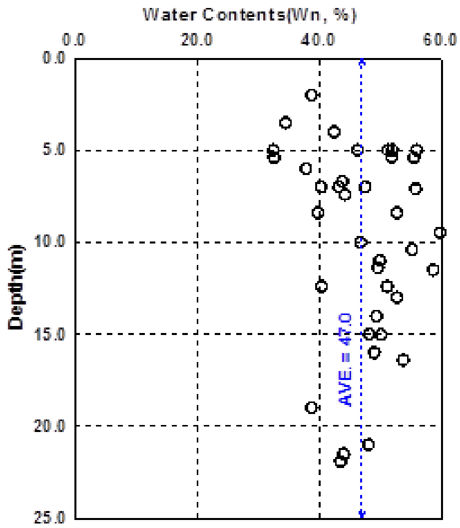

Fig. 3. Water content distribution

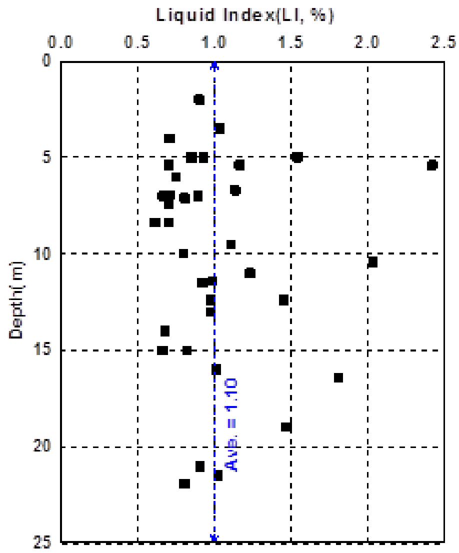

Fig. 4. Liquid index distribution

Table 2. Soil parameters

Soil parameters	Section 1	Section 2	Section 3
Liquid limit (LL,%)	21.0	21.0	31.0
Plastic index (PI, %)	25.8	31.1	23.7
Specific gravity,	2.70	2.70	2.70
Unit weight (kN/m^3)	17.0	17.0	17.5
Compression index (Cc)	0.36	0.36	0.36
Void ratio (e_o)	1.2	1.3	1.2
Coefficient of consolidation (C_v, cm^2/sec)	2.5×10^{-3}	1.0×10^{-3}	2.5×10^{-3}
Undrained shear strength (c_u, kPa)	26	24	28
Strength increase ratio (m)	0.19	0.23	0.24

pressure ratio is in the range 0.19 between 0.24, and undrained shear strength is 20~28 kN/m^2.

The presented subsoil properties are over large regions, so uniform that the settlements in this region can directly be compared to obtain a general conclusion for the deformation behavior of soft soil.

The all preconsolidation pressures are larger than 50 kPa except one value as shown in Fig. 5. The relation between OCR and soil depth can be depicted as follows:

$$\begin{aligned} \mathrm{OCR} &= 2 - 0.1Z \quad (Z < 10\mathrm{m}) \\ &= 1.0 \quad\quad\quad (Z \geq 10\mathrm{m}) \end{aligned} \tag{1}$$

The preconsolidation pressure for back analysis is determined from undrained strength and strength increase ratio, standard penetration number (N) as shown in equation (2).

$$\mathrm{Pc} = c_u/\mathrm{m} = (\mathrm{N}/0.16)/m = 6.25\mathrm{N}/m, \tag{2}$$

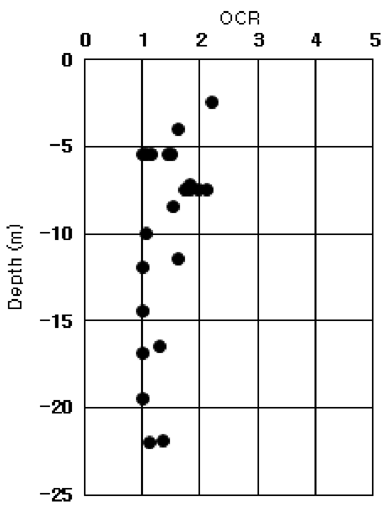

Fig. 5. Distribution of OCR

Where, m = 0.19~0.24, preconsolidation pressure, is then between 26N (kPa) and 33N (kPa). The preconsolidation pressure, of N (kPa) is applied to for conservative back analysis where N is SPT number. The preconsolidation pressure using 25N (kPa) is used for analysis. Preconsolidation of 50 kPa is employed in analysis because all of preconsolidation pressure is larger than 50 kPa.

3. Evaluation of field settlement observation

For a valuable back analysis, the conventional method to use observation and soil parameters are widely used in the geotechnical practice to estimate the primary and secondary settlement as well as the rate of consolidation. Therefore, back analysis is applied using the available long-term settlement measurements and the common practical methods to obtain representative deformation.

3.1 Back analysis procedures

The field observations were analyzed using the method developed by Asaoka (1978). This method enables to determine the final settlement S_∞ and the coefficient of consolidation C_v for a settlement-time observation. Using the "measured" C_v values, the field primary settlement S_p could be estimated. Using C_c obtained in laboratory (Table 2), the measured value S_p can be substituted into the well known equation (3) to estimate an average field compression index C_c.

$$S = S_P = \frac{H}{1+e_o}\left[c_r \cdot \log\frac{\sigma'_{vc}}{\sigma'_o} + c_c \cdot \log\frac{\sigma'_{vc}+\Delta\sigma}{\sigma'_{vc}}\right] \tag{3}$$

Where, H is the thickness of the compressible layer, e_o is the initial void ratio σ'_o is the effective initial stress and σ'_{vc} is the overburden pressure which is assumed to be equal to the preconsolidation pressure in normally consolidated deposits. The average increase of stress in the compressible layer due to the applied surface load $\Delta\sigma$ was estimated by Simpson's rule.

In addition all measurements beyond the primary consolidation time t_p were used to estimate an average field coefficient of secondary settlement C_α using the equation

$$C_\alpha = \frac{1}{H}[(s - s_p)/\log(\frac{t}{t_p})] \quad (4)$$

in which s is the measured settlement corresponding to time t (where $t > tp$) and H is the thickness of the compressible layer. In this way the field primary and secondary settlement as well as the actual rate of deformation can be determined and compared to the parameters from laboratory tests.

3.2 Results of the back analysis

By applying the methods explained previously, the field values of the thirteen locations are analyzed. By substituting these values in equations 3 and 4, respectively, the average field compression index Cc and the average field coefficient of secondary settlement could be estimated. Results from the back-analysis of the primary consolidation are summarized in Table 3.

3.3 Evaluation of the results

Similar field observation was reported by Terzaghi & Peck (1967). The high observed C_v values in the other cases are perhaps due to the multidirectional consolidation or to the existence of unknown drainage zones (stratigraphy) that may be missed in the subsurface explorations.

Data about the secondary settlement could be obtained since the consolidation time t_p for all cases was determined within the observation time. The short observation time attributes to the fact that the settlements were monitored by practicing engineers for controlling purpose and not for research purpose. On the basis of the available long-term observation a field coefficient of secondary settlement was determined using the equation (3) for all cases with an average value of C_α=0.008 of secondary compressibility according to the classification after Mesri (1973). The ratio C/C_c seems to be constant with an average value of 0.05 which is almost close to the equation of C_α/C_c = 0.04±0.01 proposed by Mesri & Choi (1984) for inorganic soft clays. The 20~40 cm of excess settlement is mainly due to the fact that when design was performed, secondary consolidation was not considered. Therefore, secondary consolidation is main factor, and the immediate settlement is considered to contribute another factor. The obvious reason that could be missed easily is the fact that construction filling did not meet the design embankment height that might cause insufficient preloading (Fig. 6 and Fig. 7).

Fig. 8 shows that when filling was finished, 126 cm settlement occurred and 24 cm further settled again with time after

Table 3. Back analysis results of the consolidation

Settlement plate	Embankment height (m)	Measured settlement (cm)	Settlement during filling (cm)	Final settlement (cm)	Residual settlement (cm)	Ratio (measured/back analysis)
SP3-7	11.14	151.7	148.2	159.6	11.4	1.02
SP3-12	11.14	177.6	150.7	173.7	23.0	1.18
SP4-1-7	10.89	148.1	146.3	169.6	23.3	1.01
SP4-2-7	12.16	161.2	147.7	164	16.3	1.09
SP5-1-3	6.91	104.1	136.7	192.8	56.1	0.76
SP5-1-4	7.18	114.5	138.1	193.6	55.5	0.83
SP5-1-5	10.93	165.2	167.7	203.9	36.2	0.99
SP5-1-6	10.35	166.9	166.9	208.3	41.4	1.00
SP5-1-10	10.48	136.5	174.3	196.9	22.6	0.78
SP5-2-1	9.61	138.8	157.2	190.1	32.9	0.88
SP5-2-3	9.29	131.1	125.7	149.5	23.8	1.04
SP5-2-8	7.68	97.8	109.2	150.5	41.3	0.90
SP5-2-15	9.20	113.4	126.9	143.2	16.3	0.89
Average	9.77	139.0	145.8	176.6	30.8	0.95

beginning of operation. It is considered that main reason is secondary consolidation, and some part is due to lack of filling showing fill elevation is lower than design load. This trend is all same to Fig. 7 and Fig. 8.

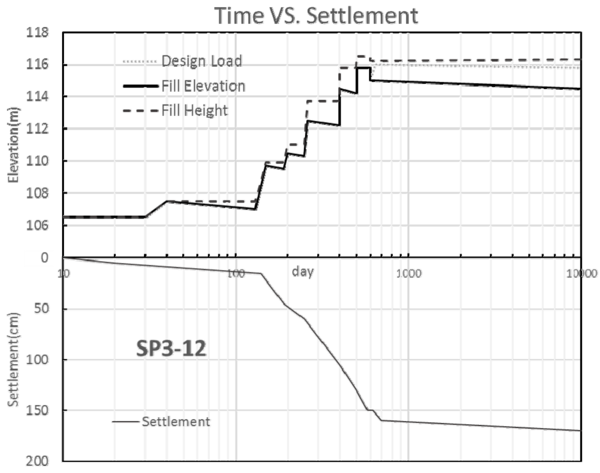

Fig. 6. Back analysis of section 3 (SP3-12)

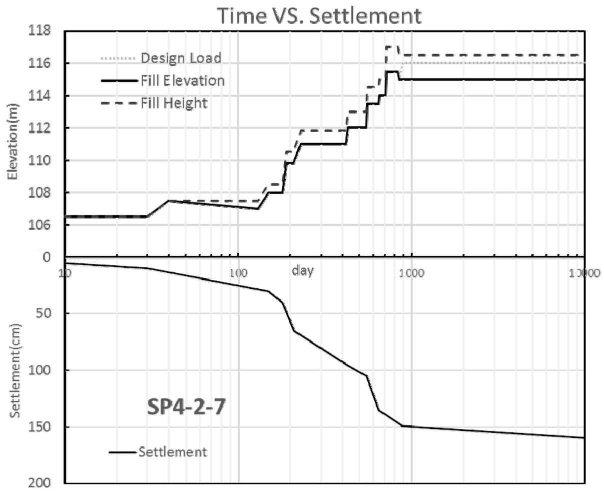

Fig. 7. Back analysis of section 4 (SP4-2-7)

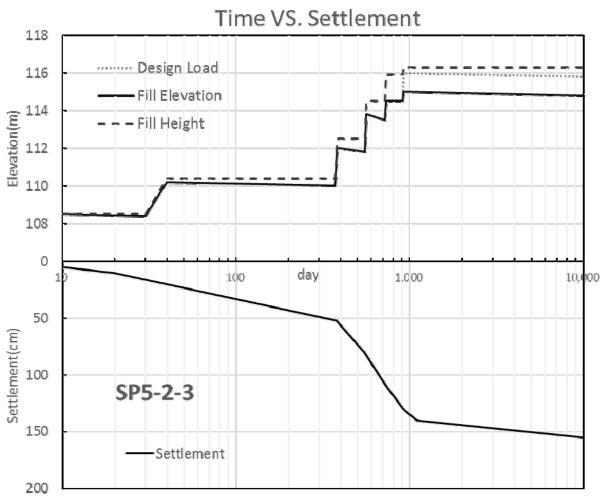

Fig. 8. Back analysis of section 5 (SP5-2-3)

4. Conclusions

A valuable back analysis of embankments on soft clay in southern region of Korea shows that actually measured settlement in this area larger than expected settlement. This enormous discrepancy is due to the passing over secondary consolidation, and the design filling did not meet to real construction filling. Immediate settlement could be subsidiary factor of excess settlement. In order to avoid this discrepancy, the constant rate of loading tests (CRL) to determine reasonable compressibility parameters for soft soils were performed .

Secondary consolidation is largely responsible for excessive settlement of 20～40 cm exceeding 10 cm of design criteria, and subsidiary to filling material itself. Current 6 mm settlement per year after construction is secondary settlement, and residual settlement is expected 7 cm for ten years from now, and 11 cm for twenty years. The further measurement should be maintained, but it seems the further settlements is controllable by field maintenance like ballast compaction.

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