View metadata, citation and similar papers at core.ac.uk

brought to you by CORE

provided by Missouri University of Science and Technology (Missouri S&T): Scholars' Mine

Missouri University of Science and Technology

Scholars' Mine

International Conference on Case Histories in Geotechnical Engineering

(1993) - Third International Conference on Case Histories in Geotechnical Engineering

02 Jun 1993, 2:30 pm - 5:00 pm

Soil Improvement Method Using Low Strength Pile with Permeability

Yoshio Wakame
Taisei Corporation, Tokyo, Japan

Masato Majima
Taisei Corporation, Tokyo, Japan

Follow this and additional works at: https://scholarsmine.mst.edu/icchge

Part of the Geotechnical Engineering Commons

Recommended Citation

Wakame, Yoshio and Majima, Masato, "Soil Improvement Method Using Low Strength Pile with Permeability" (1993). *International Conference on Case Histories in Geotechnical Engineering*. 32.
https://scholarsmine.mst.edu/icchge/3icchge/3icchge-session07/32

This Article - Conference proceedings is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 7.23

Soil Improvement Method Using Low Strength Pile with Permeability

Yoshio Wakame and Masato Majima
Leader, Geotechnical Engineering Research Group, Technology Research Center, Taisei Corporation, Tokyo, Japan

SYNOPSIS In order to reduce differential settlement and prevent heaving of surrounding ground due to fill loading, a new method using low strength pile with permeability has been devised. The pile mareríal have been composed of hydraulic granulated slag and 12~20% cement of slag. After the design method having been presented on basis of model loading test result,soil improvement have been put to practical use with sand drain method. As a result, it have been confirmed that this method has been effective for reduction settlement, prevention heaving and same drainage as sand drain method.

1. Introduction

Improving soft ground with vertical drain method may cause the differential settlement or heaving of surrounding ground due to fill loading. When structures are located adjacent to a district to be improved, or high fill is required, therefore, sand compaction pile method is frequently employed instead of vertical drain method. However, this method generates high vibration and noise at working, which gives negative effect to the surrounding areas. To replace this method, a new method to install low strength pile with permeability underground by continuous auger driving with low vibration and noise has been devised . In this method, the material mixed with hydraulic granulated slag and cement is installed into soft ground and hardened with ground water. This pile has ellective for shortening consolidation time, reducing consolidation settlement, increasing bearing capacity, protecting sliding failure, drainage and friction pile capability. Following strength and permeability tests of the pile materials (Majima and Wakame, [1]) and model loading tests of the ground installed these piles (Wakame, [2], [3], [4]), this method was put to practical use with sand drain method (Wakame and Majima, [5]). This report introduces the results of the aboves.

2. Permeability and compression strength of pile material

For the pile material, hydraulic granulated slag (here in after called slag) and cement are molded together, and hardened through the reaction with underground water. The permeability and compression strength of the pile material were estimated to vary depending on the addition ratio of cement and the relative density of slag, the relation of the permeability coefficient and unconfined compression strength of the pile material to the addtion ratio of cement and the relative density of slag were measured by laboratory test.

2.1 Sample

For the sample, ordinary portland cement is added to the slag made of Iron Mizushima Mill, Kawasaki Steel shown in Figure 1, and this material was molded fully by a mixer. The major contents of the slag are as shown in Table 1 and the grain size distribution are shown in Figure 1.

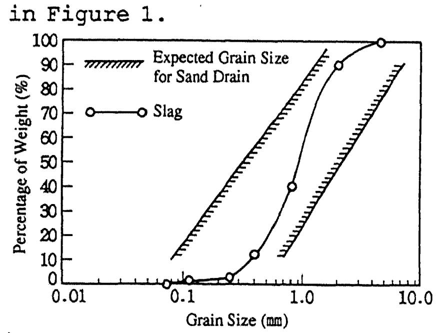


Fig.1. Grain size distribution of slag

Table 1. Major contents of slag

Contents	Percentage of weight
CaO	38−44 %
SiO_2	31−37 %
Al_2O_3	13 −19 %
MgO	3 − 8 %
Others	2 − 4 %

Table 2. Kinds of specimens

Cement Addition Ratio Cw (%)	4	6	8	10	16	20
Relative Density Dr (%)	5	5	5	5	5	5
	-	-	25	25	25	-
	40	40	40	40	40	40

2.2 Specimens

The speciments were prepared in a vinyl chloride made mold with a inner-diameter of 5cm and a height of 10cm adapting various cement addition ratio (**Cw**) and relative density (**Dr**) shown in Table 2.

2.3 Test results

2.3.1 Permeability coefficient

The permeability coefficient (**Kv**) were obtained by the falling head permeability test using a triaxial compression apparatus. The results are shown in Figure 2.

From the figure, it was found that **Kv** does not vary significantly with different **Cw** and **Dr**, staying in a range of $1.5x10^{-3}$ ~ $2.0x10^{-3}$cm/s.

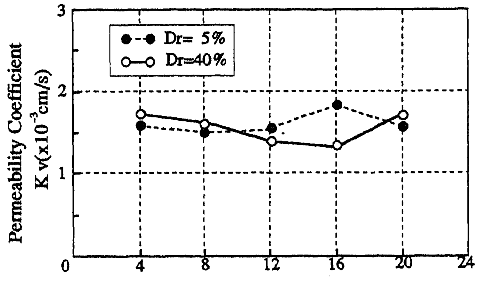


Fig.2. Permeability test results of pile material

2.3.2 Unconfined compression strength

Figure 3 shows the relationship between unconfined compression strength (**qu**) and **Cw** and **Dr**. The figure shows that the ralationship between **qu** and **Cw** is almost proportionate at the constant value of **Dr**, and that as **Dr** increases, the gradient tends to increase. With **Cw** less than 6%, the value of **qu** becomes extremely small and varies widely. Terefore, the application range of **Cw** not to be hardened was set to 8% or more.

The strength required for the pile at the practical installation is determined by **Cw**, fill loading and pile pitch. As applying cement addition ratio **Cw** more than 20% causes to lower the installation efficiency due to the pile material clogged in the auger shaft, the application range of **Cw** using this method was set to 8 ~ 20%.

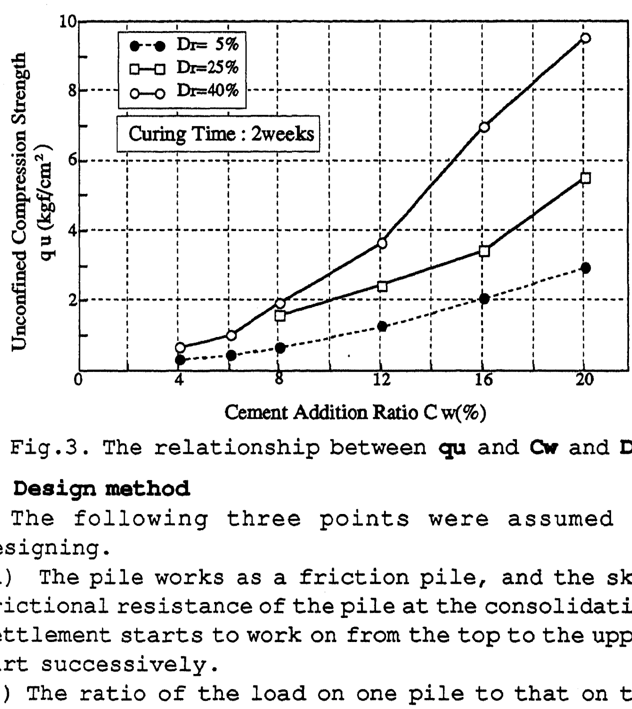


Fig.3. The relationship between **qu** and **Cw** and **Dr**

3. Design method

The following three points were assumed in designing.

(1) The pile works as a friction pile, and the skin frictional resistance of the pile at the consolidation settlement starts to work on from the top to the upper part successively.

(2) The ratio of the load on one pile to that on the effective ground area per pile is equal to the ratio of the ultimate bearing capacity of the pile to that of the ground.

(3) The load applied to the pile on the ground surface is transmitted to the ground at the total working surface of pile reactive force.

Following the above assumption (Wakame and Majima, [6]), the design method is introduced in order.

Meantime, the strength distribution of depth direction of the ground to be stabilized, the installation status of the pile, and the specifications are estimated as follows.

3.1 Ultimate bearing capacity of pile and ground

When loading is applied, the pile works as a friction pile, and the ultimate bearing capacity of one pile (**Pu**) can be expressed as Equation (1), while that of the ground (**Pd**) as Equation (2).

$$P_u=R_u+R_f=5\cdot qu\cdot A_p+\Psi\int_0^L C(x)dx \quad \text{(1)}$$

$$P_d=5.3\cdot C \quad \text{(2)}$$

Ru : End of bearing capacity of a pile
Rf : Maximum friction force of a pile
qu : Unconfined compression strength of soil near the end of pile
L : Length of a pile
Ap : Lateral sectional area of a pile
Ψ : Circumferential length of a pile
C(x): Cohesion of soil at a depth of X
C : Cohesion of soil near the ground surface

Here, the maximum value of the friction force of the pile face and cohesive soil per unit area is assumed as equal to the cohesion **C**.

3.2 Stress sharing ratio

When the uniform load **ΔP** is applied to the governing area of one pile (**a²**), the ground stress generated on the surface (**σc**), the stress generated at the pile (**σp**), and the ratio between both (**n**) can be expressed by Equation (3). The load applied to one pile (**Pp**) and that applied to the ground inside the governed area (**PG**) can be expressed with Equation (4) respectively.

$$\left.\begin{aligned} n&=\frac{\sigma_p}{\sigma_c}\\ \sigma_c&=\frac{\Delta P\cdot a^2}{a^2+(n-1)A_p}\\ \sigma_p&=\frac{n\cdot\Delta P\cdot a^2}{a^2+(n-1)A_p}\end{aligned}\right\} \quad \text{(3)}$$

$$\left.\begin{aligned} P_p&=\sigma_p\cdot A_p\\ P_G&=\sigma_c\cdot(a^2-A_p)\end{aligned}\right\} \quad \text{(4)}$$

The ratio between **Pp** and **PG** can be expressed with Equation (5). Employing the first assumption here allows to express the load sharing ratio **N** can be expressed wih Equation (6).

$$N=\frac{P_p}{P_G}=\frac{\sigma_p\cdot A_p}{\sigma_c\cdot(a^2-A_p)}=n\cdot\frac{A_p}{(a^2-A_p)} \quad \text{(5)}$$

$$N=\frac{P_p}{P_G}=\frac{5\cdot qu\cdot A_p+\Psi\int_0^L C(x)dx}{5.3\cdot C\cdot(a^2-A_p)} \quad \text{(6)}$$

Substituting Equation (6) into Equation (5) expresses the stress sharing ratio **n** as shown with Equation (7).

$$n=\frac{\sigma_p}{\sigma_c}=N\cdot\frac{a^2-A_p}{A_p}=\frac{5\cdot qu\cdot A_p+\Psi\int_0^L C(x)dx}{5.3\cdot C\cdot A_p} \quad \text{(7)}$$

3.3 Determination of pile pitch

As the pile is used for drain, the pile pitch (**a**) should satisfy Equation (8) which is determined by the degree of consolidation (**U**) and the consolidation time (**t**).

$$t \geq \frac{T_h \cdot de^2}{C_h} \quad , \quad a \leq \sqrt{\frac{t\ C_h}{T_h\ (1.13)^2}} \qquad \text{------------------ (8)}$$

de : 1.13a(square arrangement)
Th : time coefficient relating to U
Ch :vertical coefficient of consolidation

While the stress of pile (σ_p) should be larger than the unconfined compression strength of pile (F_c). The lower value of among that satisfying Equations (8) and (9) is employed as the pile pitch.

$$\left.\begin{aligned} \sigma_p &= n \cdot \frac{\Delta P + a^2}{a^2+(n-1)A_p} \leq F_c \\ a &\leq \frac{(n-1)\ A_p \cdot F_c}{n \cdot A_p - F_c} \end{aligned}\right\} \qquad \text{------------------ (9)}$$

3.4 Equivalent load surface (ref.[7])

When the load P_p (=namely $\sigma_p \cdot A_p$) is applied to the pile, the working range of friction force required (X_1) can be obtained with Equation (10) (See Figure 4). The friction force in this range (R_f) and the working surface of the combined force (X_2) can be expressed by Equations (11) respectively. The working surface (X_3) of the combined force of R_u and R_f is the equivalent load surface, which can be expressed with Equation (12).

$$P_p = 5 \cdot qu \cdot A_p + \Psi \int_0^{X_1} C(x)dx \qquad \text{------------------ (10)}$$

$$\left.\begin{aligned} R_f &= \Psi \int_0^{X_1} C(x)dx \\ X_2 &= \frac{\Psi \int_0^{X_1} X \cdot C(x)dx}{\Psi \int_0^{X_1} C(x)dx} \end{aligned}\right\} \qquad \text{------------------ (11)}$$

$$X_3 = \frac{R_f \cdot X_2}{R_f + R_u} \qquad \text{------------------ (12)}$$

3.5 Increased stress in the ground

Assuming that the load applying to the pile (P_p) works on equivalent load surface of ground uniformly, the uniform load (ΔP_1) can be expressed by Equation (13) (Figure 5(a)). While assuming that the P fill load ($\Delta Pa^2 - P_p$) works on the ground surface uniformly, the uniform load (ΔP_2) can be expressed by Equation (14). Consequently, the distribution of the increased stress in the ground is as shown in Figure 5(c).

$$\Delta P_1 = \frac{P_p}{a^2 - A_p} \qquad \text{------------------ (13)}$$

$$\Delta P_2 = \frac{\Delta P\ a^2 - P_p}{a^2 - A_p} \qquad \text{------------------ (14)}$$

3.6 Amount of consolidation settlement

For the calculation of the amount of consolidation settlement, thick of layer (**H**) is divided in two layers, H_1 and H_2, at the equivalent load surface shown in Figure 5, and the increased ground stress of each layer is used. When the amount of settlement of H_1 and H_2 layers is expressed with S_1 and S_2 respectively, the total amount of settlement is $S=S_1+S_2$, and the settlement distribution in the depth is as shown in Figure 6.

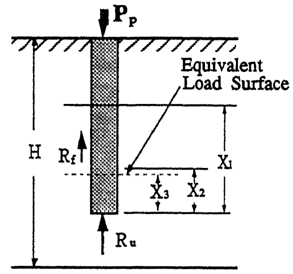


Fig.4. Equivalent load surface

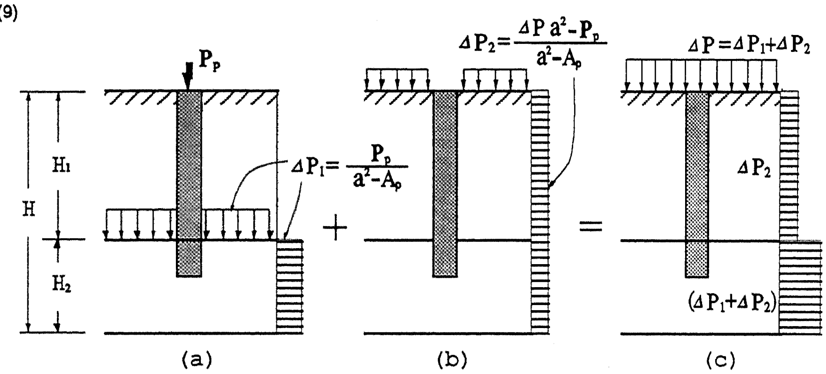


Fig.5. Distribution of increased stress in the ground

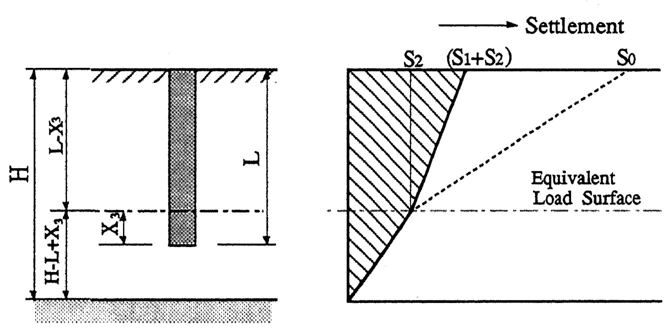


Fig.6. Settlement distribution in the depth

4. Practical example

At the construction work of a housing development in the suburb of Tokyo, the ground and roads surrounding the buildings which were supported by PC-piles had been settled for a long period by filling, a ground stabilization work was applied to the entire site previously.

4.1 Soil profile and soil constant

Figure 7 shows the soil profile of this ground and the soil constant. The layer thickness of each layer is occupied by 1.5m of silty sand and 4.0m of humic soil approximately showing almost constant value, however, the thickness of cohesive soil is in a range 1~3m. The humic soil showed high void ratio and water content indicating the state where the roots of plants have not been decomposed.

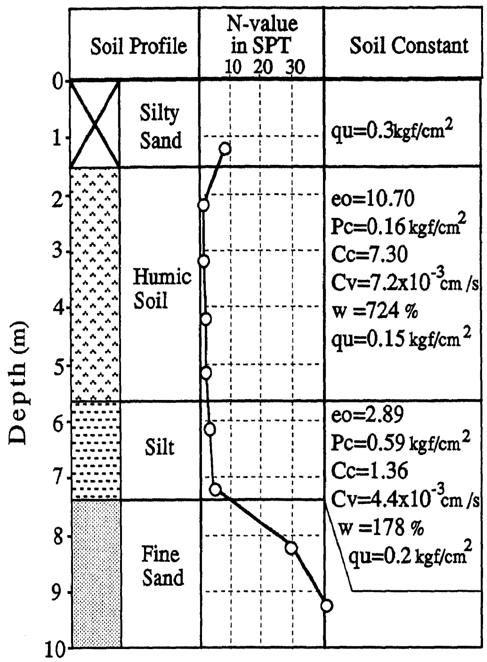


Fig.7. Soil profile and soil constant

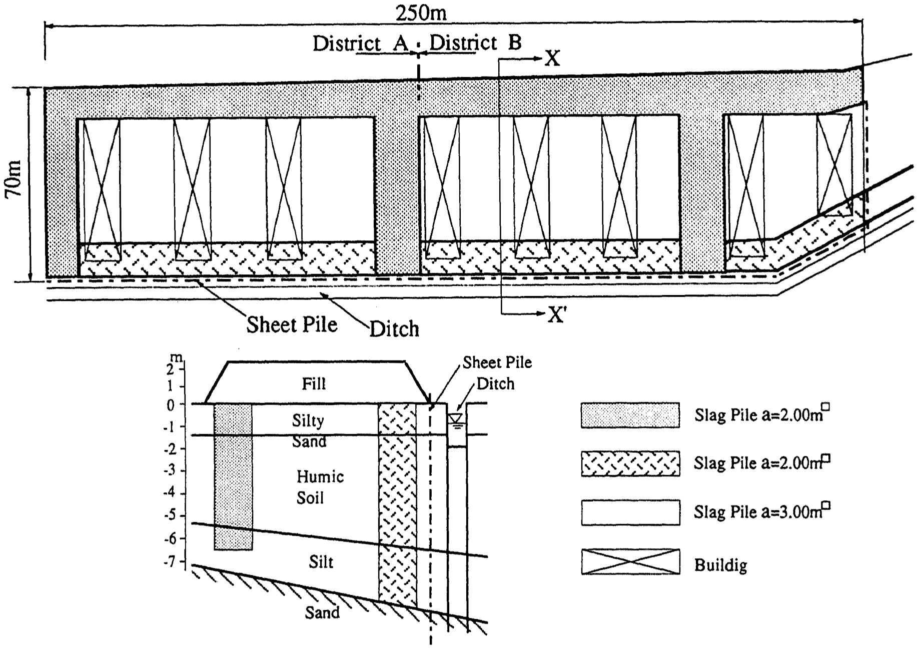


Fig.8. Location of buildings and stabilization

4.2 Outline of work

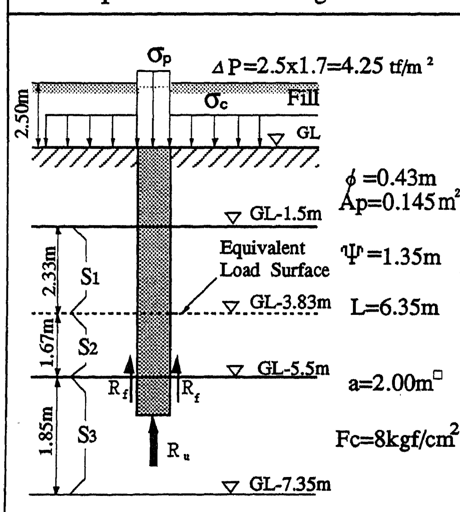
As shown in Figure 8, 8 buildings were constructed by filling the site of 18,000m^2 approximately for a height of 1~2.5m. For stabilization, slag piles, sand piles and sheet piles were installed by dividing areas as shown in Figure 11. The slag piles were used to the area of major roads (where underground pipe is installed) to bear the underground pipes after consolidation settlement to reduce residual settlement.For other areas, the sand piles were driven, and the filling was transfered from District A to District B. The numerical amount for each work is shown in Table 3.

At the installing of the slag piles and sand piles, the auger with less noise was used to minimize the effect to the adjacent residences.

Table 3. The Amount for Each Work

Kinds of Works	Amount
Sand Mat : 50cm thick	8,930m^3
Filling : about 250cm thick	District-A : 23,000m^3 District-B : 14,000m^3
Slag Pile : 43cm in diameter	9,905m
Sand Pile : 43cm in diameter	12,037m
Sheet Pile : Type-III	5,440m

Table 4. Specification of slag pile and calculation result of each item

Specification of Slag Pile	Item	Calculation Result
	Bearing Capacity of a Pile	$P_u=5\cdot qu\cdot A_p+\Psi\int_0^L C(x)dx=9.38tf$
	The Ratio between P_P and P_G	$N=\frac{P_P}{P_G}=\frac{5\cdot qu\cdot A_p+\Psi\int_0^L C(x)dx}{5.3\cdot C\cdot(a^2-A_p)}=0.589$
	The Ratio between σ_P and σ_C	$n=\frac{\sigma_P}{\sigma_C}=N\cdot\frac{a^2-A_p}{A_p}=15.5$
	The Load Applied to a Pile	$P_P=\sigma_P\cdot A_p=\frac{A_p n\,\Delta P\cdot a^2}{a^2+(n-1)A_p}=6.1tf$
	Equivalent Load Surface	$X_3=\frac{R_f\cdot X_2}{R_f+R_u}=0.0252m$
	Settlement	$S_1=0.359m$ $S_2+S_3=0.467+0.138=0.605m$ $\Sigma S=0.954m$ $\Sigma S_0=1.117m$
	Reduction Ratio in Settlement	$\frac{\Sigma S_0-\Sigma S}{\Sigma S_0}\times 100=13.5\%$

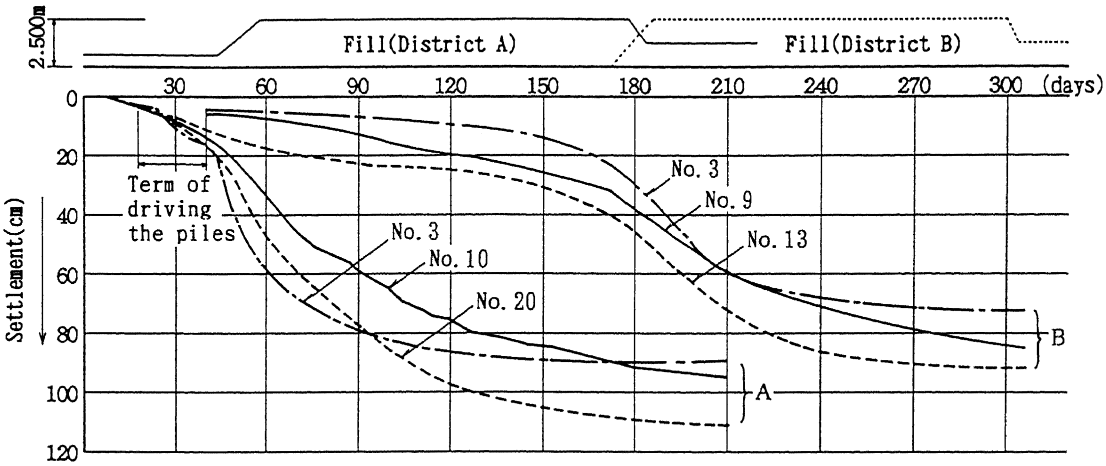


Fig.9. Examples of time-settlement curves at the ground surface

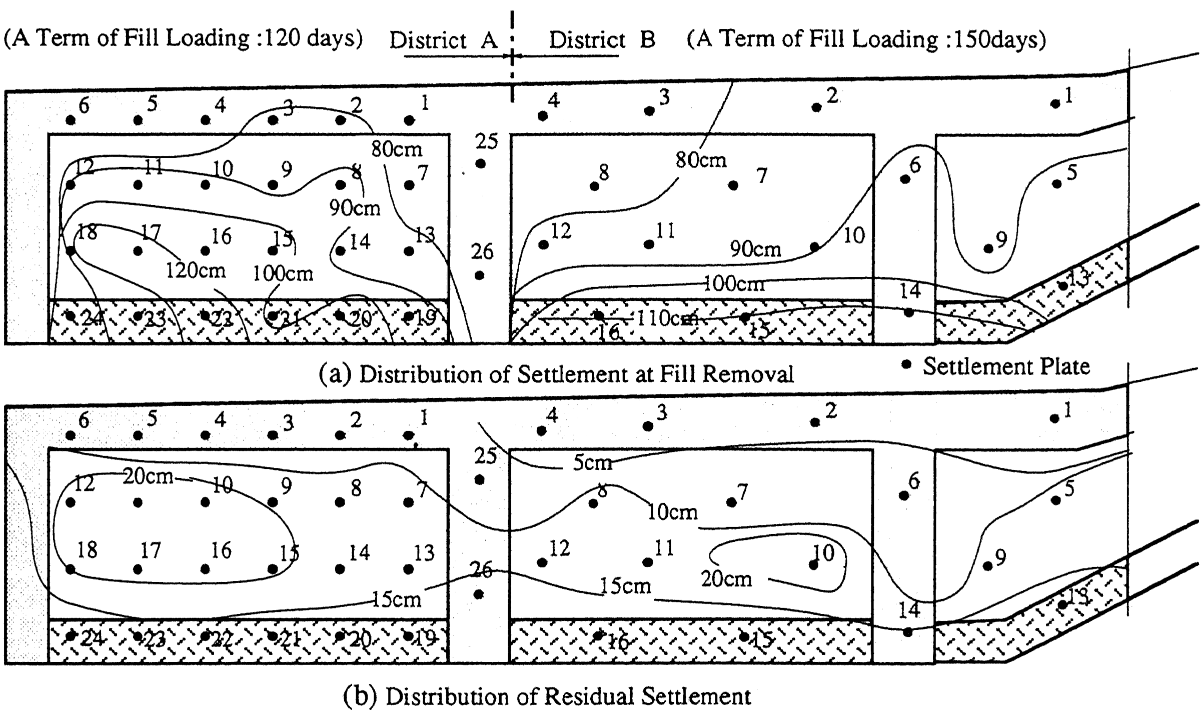


Fig.10. The distribution of amount and residual settlement at fill removal

l.3 **Stabilization plan**

Sand pile pitch was determined so as to obtain the legree of consolidation more than 95% at the date ifter leaving the fill alone for 120 days. Assuming that the consolidation coefficient of the humic soil (**Cv**) is $4.35\times10^{-3}cm^2/sec$, the pitch forms a square arrangement. As the design method of slag piles was already intruduced, only the calculation result is shown in Table 4.

l.4 **Measurement results**

l.4.1 **Amount of settlement**

The settlement of the ground surface caused by the load of fill is shown in Figure 10. The measurement

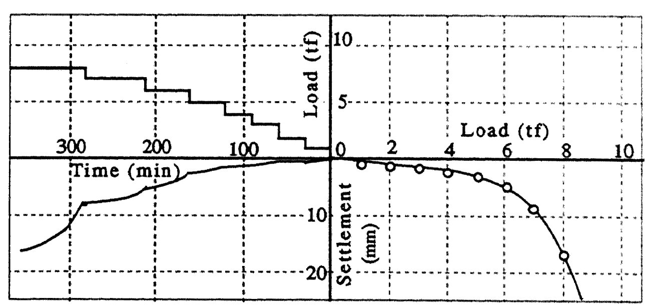


Fig.11. An example of the vertical loading test result of a pile

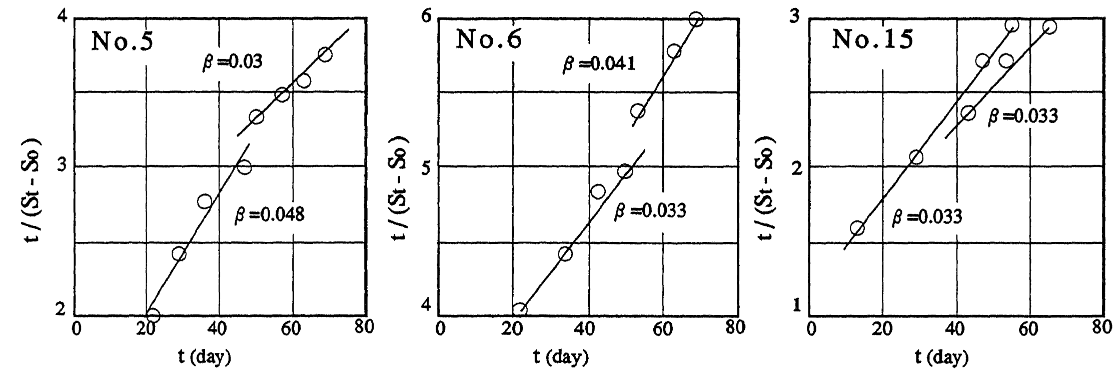


Fig.12. The β-values obtained from the time-settlement curves using hyperbola method

was conducted by installing settlement plate gauge at the marked points. The state of the settlement was grasped by installing at 26 spots in District A, and 16 spots in District B. Figure 9 shows an example of the relationship between the settlement and the elapsed time. The District B was filled after 150 days from the drain casting. Figure 9 shows the amount of settlement at fill removal, and the distribution of residual settlement. The amount of the residual settlement was obtained by sabtracting the amount of settlement at fill removal from the final settlement being obtained by hyperbola method using the measured value.

As an average 1.5m of fill was removed, a 2~3 cm of the rebound was generated.

4.4.2 Bearing capacity of the slag pile

For the bearing capacity of the slag pile, a vertical loading test was applied after fill removal to the three piles near the settlement measuring point 3 in District B. An example of one of the results is shown in Figure 11.

4.5 Consideration

Since the pile pitch and the thickness of the poor ground layer differ, the effect to reduce settlement could not been obtained directly from the measured values. Judging from the distribution of residual settlement shown in Figure 10, almost points in the slag pile district display a consolidation degree of more than 95% at the time of the fill removal. Figure 12 shows β values obtained from the settlement-time curves for the measuring points No.5, 6 and 15 in District A using the hyperbola method (Miyagawa, [8]). Compared with the value at the measuring point No.6 (slag pile district), the β values at the measuring points No.5 and No.15 are moving to the smaller despite of the longer period to leave the fill alone, showing the trend to promote settlement. From the above trend, the effect to reduce settlement by the slag pile can be expected. The bearing capacity of 8~9 tons carried by the slag pile would thus be large enough to protect underground piles or the like against the residual settlement.

5. Conclusion

The following items have been clarified regardingthe method using low strength pile with permeability.

1. The pile added a 12%~ 20% (weight ratio) of ordinary portland cement to the hydraulic granulated slag obtains the unconfined compression strength of 3kgf/cm² and 5kgf/cm² and permeablity of $2x10^{-3}$cm²/s.
2. The excellent permeability of the pile is effective to reduce consolidation time, and the effect of friction force of the pile is effective to lower consolidation settlement.
3. As the mean load (σ) increases, the stress shearing ratio (n) tends to decrease, and the value of the stress shearing ratio (n) obtained by the computing equation mostly agrees with that obtained by the experiment under the load condition where both pile and ground reach the ultimate bearing capacity.

REFERENCES

[1] Majima,M. and Wakame,Y. (1979), "The Soil Improvement Method Using Low Strength Pile with Permeability (Permeability and Compressibility of Pile Material)"Proceedings, 14th Japan Natinal Conf. on SMFE, pp.1601-1604.

[2] Wakame,Y.(1982)"Experimental Study on Bearing Mechanism of Composite Ground-Part 1", Procedings, 17th Japan National Conf. on SMFE, pp.2409-2412.

[3] Wakame,Y.(1983)"Experimental Study on Bearing Mechanism of Composite Ground-Part 2", Procedings, 18th Japan National Conf. on SMFE, pp.1475-1476.

[4] Wakame,Y.(1984)"Experimental Study on Bearing Mechanism of Composite Ground-Part 3", Procedings, 19th Japan National Conf. on SMFE, pp.1683-1684.

[5] Wakame,Y. and Majima,M. (1979), "The Protection Method on Deformation of Surrounding Ground due to the Embankment (The Soil Improvement Constraction Using Slag Pile Method)", Proceedings, 14th Japan National Conf. on SMFE, pp.1605-1608.

[6] Wakame, Y. and Majima, M. (1980),"The Soil Improvenent Method Using Low Strength Pile with Permeability (Design Method)", Proceedings, 15th Japan National Conf. on SMFE, pp.1685-1688.

[7] The Architectural Institute of Japan, (1988)"Recommendations for Design of Building Foundations", pp.369-371.

[8] Miyagawa, I.(1961)"Soft Ground and Enbankment",Design Method of Soil and Foundaiton (Part-3), The Japanese Society of SMFE, pp.178-181.