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A Strength Estimation Formula for Backfill Soil Following Existing Pile Removal and Its Validation Through Field Testing

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ABSTRACT: The mechanical properties of backfill soil, which is generated during existing pile removal and backfilling using the casing cyclic shear removal method, were evaluated through unconfined compression tests and ultrasonic propagation velocity tests in this study. The experimental results showed that the relationship between unconfined compressive strength and *S*-wave velocity exhibited similar characteristics to those of liquefied stabilized soil. An effective cement-to-water ratio concept was proposed by considering the water retention characteristics of bentonite, which is used as backfill material. A strength estimation formula for backfill soil was subsequently developed based on the relationship between unconfined compressive strength and the effective cement-to-water ratio. Furthermore, to validate the proposed strength estimation formula, a field test was conducted, and backfill soil with a target unconfined compressive strength range was designed using this formula. The soil investigation results following field testing revealed that the backfill soil area exhibited *N*-value distribution patterns comparable to those of the surrounding natural ground. Additionally, undisturbed samples obtained from the lower section of the backfill soil demonstrated uniform unconfined compressive strength that closely approximated the lower limit of the estimated strength range.

KEYWORDS: Existing pile, Removal, Backfill soil, Unconfined compressive strength

1 INTRODUCTION

In Japan, urban reconstruction projects have increased in recent years, as buildings constructed during the high economic growth period from 1955 to 1973 are gradually aging. The existing piles of these aging buildings, which interfere with the installation of new piles for reconstruction, must be removed. The common removal methods in Japan are the all-casing crushing removal method and the casing cyclic shear removal method. In the all-casing crushing removal method, the existing piles and soil within the casing are crushed and completely removed, and the removal boreholes are backfilled with liquefied stabilized soil. However, this method has high costs and requires a large drilling machine and extensive construction space. In contrast, the casing cyclic shear removal method is the most widely adopted, as it has fewer site restrictions and reduces costs by reusing the drilling mud generated during the removal process. After removing the existing piles, backfill material (such as soil, cement slurry, or cement-bentonite slurry etc.) is injected and mixed uniformly with the drilling mud for backfilling (Shimada et al. 2020).

The quality and efficiency of new pile installations are adversely affected by the strength of backfill soil in the borehole. When the strength is insufficient compared to the surrounding natural ground, borehole wall collapse or drift may occur during new pile installation. Conversely, excessive strength can prolong excavation time for new pile boreholes. Therefore, it is critical to ensure the appropriate strength of backfill soil, especially for the casing cyclic shear removal method. Previous research has focused on developing new backfill materials and investigating mix proportions, material segregation, and the mechanical properties of backfill soil through laboratory tests (Inazumi, 2024). However, at construction sites, the physical properties of drilling mud in boreholes after pile removal using the casing cyclic shear removal method are influenced by various factors, including drilling water volume, groundwater level, and the soil properties of natural ground. This variability makes it difficult to design appropriate backfill material mix proportions. Furthermore, uniform mixing of the drilling mud with the injected backfill material is essential for the backfill soil to achieve the target strength (Furugaichi et al. 2018).

In this study, the mechanical properties of simulated backfill soil generated using the casing cyclic shear removal method were evaluated through unconfined compression tests and ultrasonic propagation velocity tests. Based on the test results, we newly propose an effective cement-to-water ratio concept that considers the water retention characteristics of bentonite. A strength estimation formula for backfill soil was subsequently developed based on the relationship between unconfined compressive strength (q_u) and the effective cement-to-water ratio. To validate its application, a field test was conducted, and backfill soil with a target strength range was designed using the proposed formula. Additionally, the mechanical properties of the backfill soil were evaluated through soil investigation.

2 CASING CYCLIC SHEAR REMOVAL METHOD

The casing cyclic shear removal method generally follows the steps shown in Figure 1. First, a pit is excavated to expose the existing pile head (Figure 1a). Then, a casing with a larger diameter than the existing pile rotates and advances downward while discharging high-pressure water to the pile tip (Figures

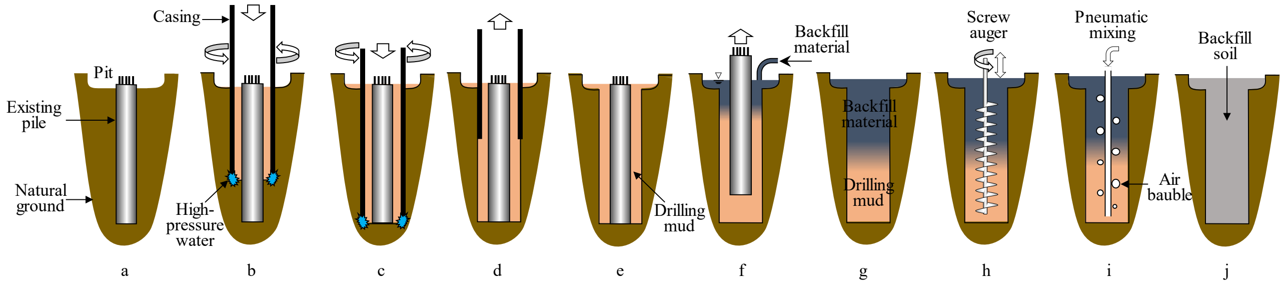


Figure 1. Existing pile removal and backfilling using the casing cyclic shear removal method.

1b and 1c). The casing is removed (Figure 1d) and the ground between the casing and the existing pile becomes drilling mud (Figure 1e). The existing pile is gradually removed using a wire, while backfill material is continuously injected into the borehole to maintain the groundwater level and ensure borehole wall stability (Figure 1f). Once the existing pile is completing removed, the drilling mud settles in the lower section and the backfill material settles in the upper section of the borehole (Figure 1g). The backfill material and the drilling mud are then uniformly mixed using a screw auger or pneumatic mixing (Figures 1h to 1j).

3 TARGET STRENGTH OF BACKFILL SOIL

Considering the re-excavation for a new pile borehole, the backfill soil must have sufficient strength to be self-standing while avoiding excessive strength compared to the surrounding natural ground. According to the Tokyo Metropolitan Government Bureau of Construction (2023), the q_u of liquefied stabilized soil used as backfill material is specified as 130 to 550 kN/m². In this study, the target strength range for backfill soil was set within this specified range.

4 PROPOSAL OF STRENGTH ESTIMATION FORMULA BASED ON LABORATORY TESTS

4.1 *Test materials*

The test materials used in the laboratory mix design tests were water, Toyoura sand, kaolin clay, cement, and bentonite. The densities of these materials are shown in Table 1. Toyoura sand is a Japanese standard sand that has been widely used in geotechnical laboratory tests. The cement was Portland blast-furnace slag cement (Type B) in accordance with Japanese Industrial Standard (JIS R 5211: 2009). Four types of bentonite with different viscosities were used: B1, B2, B3, and B4.

To evaluate the viscosities of bentonite, funnel viscosity tests were conducted on bentonite slurries prepared by mixing bentonite and water at mass concentrations of 1.5% to 12.0%. Figure 2 shows the relationship between mass concentration and funnel viscosity values (FV-value) for B1 to B4. As shown in the figure, the FV-value of the bentonite slurries generally increased with increasing mass concentration. Comparison among bentonite types indicated that viscosity decreased in the order B1 > B2 > B3 > B4.

Table 1. Densities of test materials.

	Water	Cement	Bentonite	Toyoura sand	Kaolin clay
Symbol	W	C	B	S_s	S_c
Density (g/cm^3)	1.00	3.04	2.60	2.65	2.75

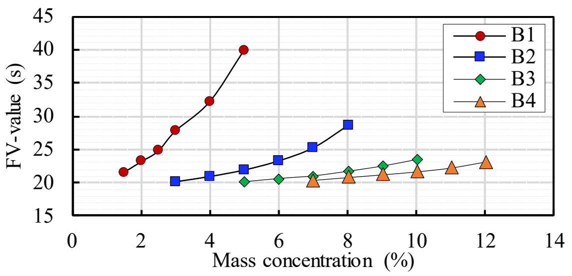


Figure 2. Comparison of funnel viscosity values obtained by different types of bentonite.

4.2 *Laboratory Tests*

Laboratory mix design tests for preparation of backfill soil specimens were conducted in accordance with the Japanese geotechnical society standard (JGS 0821-2020) through the following testing procedure: (1) simulated drilling mud with a density of 1.25 g/cm³ was prepared by mixing water with a mixture of Toyoura sand and kaolin clay at a mass ratio of 1:1; (2) bentonite was gradually added to the drilling mud and mixed for 3 minutes using a hand mixer; (3) cement was then added and mixed for another 3 minutes to produce fresh backfill soil samples; and (4) the freshly mixed samples were placed in cylindrical molds (50 mm diameter × 100 mm height) and cured at 20°C to prepare backfill soil specimens for 28-day and 56-day curing periods.

Following preparation of fresh backfill soil samples, the flow value tests (NEXCO Testing Methods, 313-1999) were conducted to evaluate the flow value (F-value), which represents the flowability of the backfill soil samples. Bleeding ratio tests (JSCE-F, 522-2018) were conducted on the fresh backfill soil samples to determine the bleeding ratio (B_r), which indicates the susceptibility to material segregation. Additionally, unconfined compression tests (JIS A 1216: 2020) were conducted on 28-day and 56-day specimens, respectively. Ultrasonic wave tests (JGS 2564-2020) were conducted on the 56-day specimens.

Table 2 shows the mix proportions of all specimens. The specimens had the unit weight of cement (C) of 128 to 135 kg/m³ and a constant cement-water ratio (C/W) of 0.2, where W is the unit weight of water. To investigate the effects of bentonite type on q_u values of backfill soil, four types of bentonites were employed, with the unit weight of bentonite (B) of 14 to 143 kg/m³ and bentonite mass concentrations ($B/(B+W)$) of 1.7% to 17.0% relative to the water in the simulated drilling mud.

Table 2. Mix proportions of all specimens.

Names	Mix proportion (kg/m^3)					C/W	$B/(B+W)$ %
	W	C	B	S_s	S_c		
B1-1	810	135	14	189	189	0.2	1.7
B1-2	806	134	28	188	188	0.2	3.3
B1-3	802	134	42	187	187	0.2	4.9
B1-4	797	133	55	186	186	0.2	6.5
B2-1	808	135	23	188	188	0.2	2.8
B2-2	800	133	46	186	186	0.2	5.4
B2-3	793	132	69	185	185	0.2	8.0
B2-4	786	131	92	183	183	0.2	10.5
B3-1	809	135	17	188	188	0.2	2.1
B3-2	807	134	25	188	188	0.2	3.0
B3-3	799	133	50	186	186	0.2	5.9
B3-4	791	132	75	184	184	0.2	8.7
B3-5	783	131	100	182	182	0.2	11.3
B3-6	778	130	116	181	181	0.2	13.0
B3-7	770	128	143	179	179	0.2	15.7
B4-1	766	128	157	178	178	0.2	17.0
B4-2	775	129	128	180	180	0.2	14.1
B4-3	789	132	82	184	184	0.2	9.4
B4-4	809	135	19	188	188	0.2	2.3

4.3 *Test results*

Table 3 shows the laboratory test results of all specimens. Figures 3 and 4 show the *F*-value and B_r of all fresh backfill soil samples, obtained from the flow tests and the bleeding tests respectively. Regardless of bentonite type, the general trend indicated that F-values decreased with increasing bentonite mass concentration, with a corresponding decrease in the flowability of the backfill soil samples. Similarly, the B_r decreased with increasing bentonite mass concentration, thereby making material segregation less likely. This trend is attributed to the water retention characteristics of bentonite, a clay mineral with significant adsorptive properties. Furthermore, regarding the influence of bentonite type, both the *F*-value and B_r of backfill soil samples increased sequentially from B1 to B4, corresponding to viscosity characteristics of the respective bentonites.

Figure 5 shows the relationship between q_u and deformation modulus (E_{50}) for all specimens, along with the q_u-E_{50} relationships for the liquefied stabilized soil made from various soil (Liquefied Stabilized Soil Utilization Technology Manual, 2008). The q_u-E_{50} relationships of backfill soil specimens exhibit comparable trends to those of liquefied stabilized soil. This correspondence suggests similar mechanical characteristics between the backfill soil and liquefied stabilized soil.

Figure 6 shows the relationship of q_u-*S* wave velocity (V_S) for all specimens, along with the q_u-V_S relationships for previously studied liquefied stabilized soil (Furugaichi et al. 2014). Here, V_S was obtained from ultrasonic wave tests. The relationship between q_u and V_S of backfill soil in this study can similarly be represented by a power approximation curve, as observed for the liquefied stabilized soil, which indicates that the simulated backfill soil has mechanical properties similar to those of the liquefied stabilized soil.

Table 3. Laboratory test results.

Names	*F* (mm)	B_r (%)	q_u (kN/m^2)		E_{50} (MN/m^2)		V_s (m/s)	*C/W'*	
			28-day	*56-day*	*28-day*	*56-day*		*28-day*	*56-day*
B1-1	500	4.0	151	205	62	63	326	0.212	0.197
B1-2	370	1.8	150	244	56	83	349	0.294	0.241
B1-3	230	0.1	169	300	49	50	336	0.480	0.311
B1-4	200	0.0	219	400	84	52	365	1.339	0.440
B2-1	440	3.8	142	189	31	43	313	0.167	0.184
B2-2	350	2.4	182	207	62	45	316	0.167	0.207
B2-3	270	2.3	255	311	44	62	361	0.167	0.236
B2-4	210	0.0	263	378	63	85	393	0.167	0.275
B3-1	490	6.8	95	200	68	86	343	0.184	0.175
B3-2	490	4.3	169	201	27	56	333	0.193	0.179
B3-3	370	1.4	173	250	72	50	335	0.230	0.194
B3-4	290	0.0	300	353	80	122	370	0.286	0.211
B3-5	240	0.0	318	439	133	56	402	0.381	0.233
B3-6	250	0.0	212	419	155	118	417	0.489	0.250
B3-7	190	0.0	521	602	248	131	503	0.928	0.284
B4-1	230	0.0	370	1092	201	244	640	0.289	0.330
B4-2	300	0.0	-	845	134	475	528	0.252	0.277
B4-3	410	1.9	206	406	117	194	398	0.212	0.223
B4-4	590	10.3	125	205	97	145	358	0.175	0.177

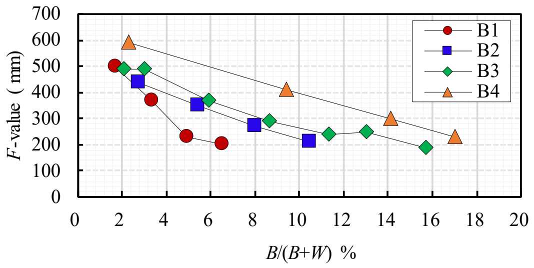


Figure 3. Comparison of the flow values obtained from backfill soil samples.

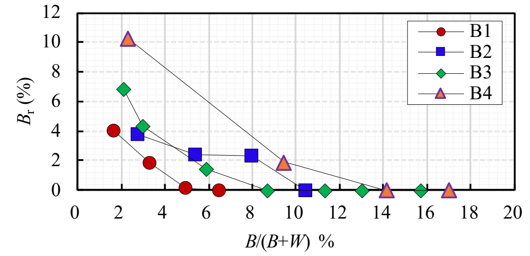


Figure 4. Comparison of the bleeding ratios obtained from backfill soil samples.

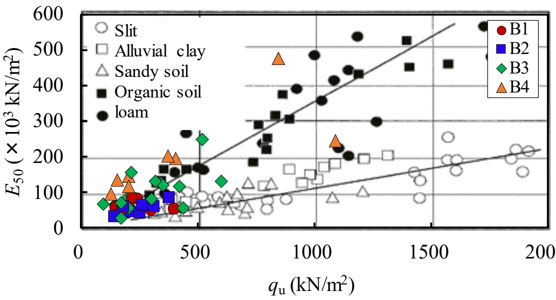


Figure 5. Comparison of q_u-E_{50} relationships obtained from backfill soil and liquefied stabilized soil.

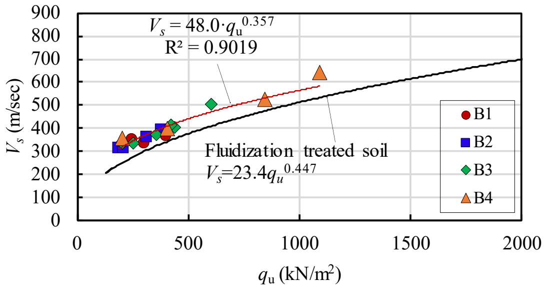


Figure 6. Comparison of q_u-V_S relationships obtained from backfill soil and liquefied stabilized soil.

4.4 *Strength estimation formula*

The cement-to-water ratio (*C*/*W*) has been widely used for designing concrete mixes and controlling concrete strength due to the linear relationship between *C*/*W* and concrete strength established by Lyse (1931). In this study, all specimens had a fixed *C*/*W* of 0.2. Because bentonite has significant water adsorption properties, we introduced the effective cement-to-water ratio (*C*/*W'*) as a more appropriate parameter for estimating the strength of backfill soil. The effective unit weight of water (*W'*) is calculated via Equation (1), where α is a constant depending on bentonite types, and α×*B* represents the water absorbed by bentonite in the backfill soil.

$$W' = W - \alpha \times B \tag{1}$$

A strength estimation formula for backfill soil was subsequently developed based on the linear relationship between q_u and C/W', as shown in Equation (2), where b and c represent the regression coefficient and constant term, respectively.

$$q_u = b \times C/W' + c \tag{2}$$

Figure 7 shows the relationship between C/W' and q_u for all specimens, with the corresponding C/W' values listed in Table 3. Table 4 shows the respective α values for different bentonite types with 28-day and 56-day curing periods. For each curing period, the α values for each bentonite type were determined using the GRG nonlinear algorithm, which maximized the coefficient of determination (R^2) for the linear regression between C/W' and q_u, with the constraint that the relationship passes through the origin ($C/W' = 0$, $q_u = 0$).

Table 4. Material constant α for strength estimation formulas.

	B1	B2	B3	B4
α (28-day)	5.2	4.5	3.9	3.1
α (56-day)	5.2	2.8	3.0	3.7

The strength estimation formulas are proposed in Equations (3) and (4) for 28-day and 56-day specimens, respectively.

$$q_u = 795.31\,C/W' + 0.6815 \tag{3}$$

$$q_u = 1811.9\,C/W' - 94.89 \tag{4}$$

The strength estimation formulas assume that bentonite retains water from the backfill soil, reducing the water available for cement hydration and thereby increasing backfill soil strength. The α is expected to correlate with the physical properties of bentonite, with higher viscosity corresponding to larger α values. However, since this correlation was observed only for 28-day specimens, further investigation is required. It should be noted that these strength estimation formulas were developed without considering the effects of soil properties. Given that the unit weight of soil in the backfill soil is relatively small compared to ground improvement applications, the proposed formulas can estimate the approximate strength range using W', C, and B within the borehole.

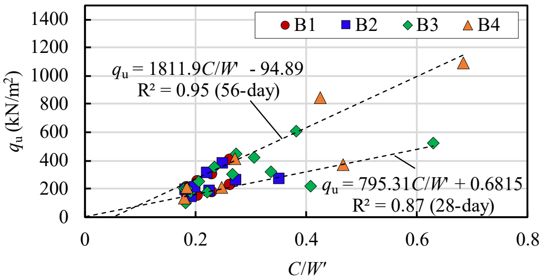


Figure 7. Strength estimation formulas for backfill soil (28-day and 56-day).

5 FIELD TEST

To validate the proposed strength estimation formulas, a field test of existing pile removal and backfilling was conducted at a construction site. The optimal mix designs for backfill materials were determined using these formulas.

5.1 *Overview of Existing Pile and Surrounding Natural Ground*

Figure 8 shows the relative positions between the existing and new piles along with their respective dimensions. The existing pile is a precast concrete pile with a diameter of 350 mm and a length of 7.0 m. The new pile is a cast-in-place concrete belled pile, positioned with an edge-to-edge spacing of 0.175 m from the existing pile. Given the accuracy requirements for the new pile installation, removal of the existing pile was necessary due to the high risk of interference with the new pile. Figure 9 shows the depth location of the existing pile and the soil boring log of the surrounding natural ground. The surface layer comprises a loam layer extending from the ground level (G.L.) to approximately G.L.-1.9 m, followed by a tuffaceous clay layer from approximately G.L.-1.9 m to G.L.-3.2 m, and below G.L.-3.2 m, a sequence of silty fine sand, gravel-bearing fine sand, and silty fine sand layers in descending order. Moreover, the groundwater level (W.L.) within the construction site is relatively low.

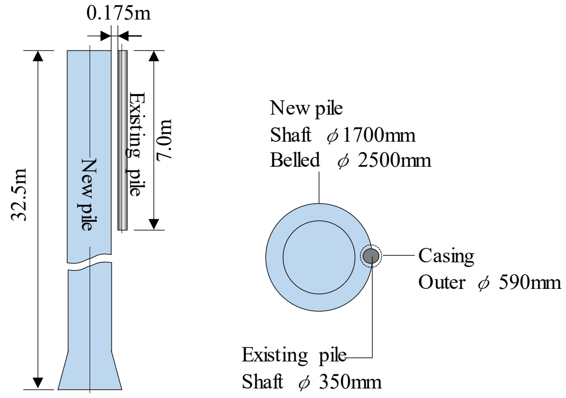


Figure 8. Relative positions between the existing pile and the new pile.

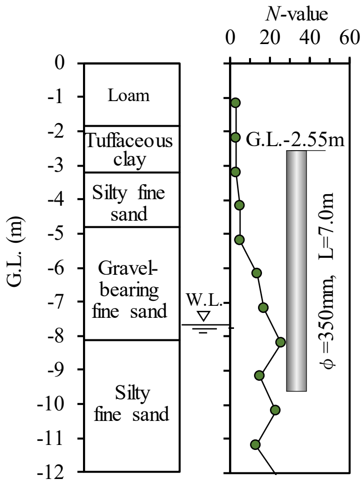
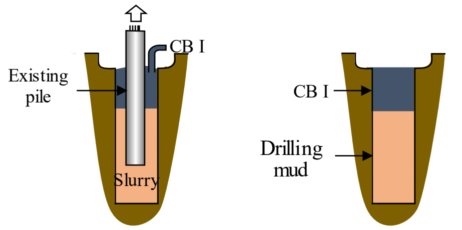


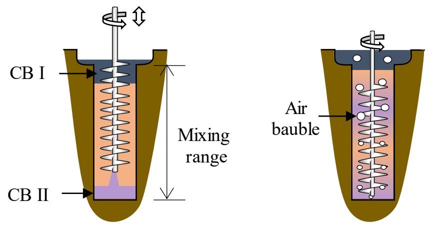
Figure 9. Soil boring log of surrounding natural ground and depth location of the existing pile.

5.2 *Removal and Backfilling/Mixing procedures*

Figure 10 shows procedures for the existing pile removal, backfilling, and mixing conducted in the field test. The pile removal was conducted using the casing cyclic shear removal method as described in Figure 1. Two types of cement-bentonite slurry, designated as CB I and CB II, were prepared as backfill materials using a mixing plant. Considering the relatively low groundwater level at the site, the pile was removed while CB I was continuously injected from the top of the borehole to maintain a constant drilling mud level, thereby preventing potential borehole wall collapse (Figure 10a). After completion of pile removal and CB I injection, it was assumed that CB I occupied the upper section of the borehole while drilling mud remained in the lower section (Figure 10b). Subsequently, a screw auger was inserted into the lower portion of the borehole, and vertical mixing was performed while CB II was injected from the auger tip (Figure 10c). Finally, pneumatic mixing was conducted from the bottom of the borehole for approximately 20 minutes (Figure 10d).



a. Pile removal b. After fishing removal



c. CB II injection d. Pneumatic mixing

Figure 10. Existing pile removal, backfilling, and mixing procedures at the field test.

5.3 *Field Mix Proportion*

Since the bentonite used in this field test exhibited similar viscosity to bentonite B4, the strength estimation formula for bentonite B4 was used to predict the backfill soil strength for 28-day and 56-day curing periods via Equations (5) and (6), respectively.

$$q_u = 795.31\, C/(W - 3.1 \times B) + 0.6815 \tag{5}$$

$$q_u = 1811.9\, C/(W - 3.7 \times B) - 94.89 \tag{6}$$

To achieve the target strength (130 to 550 kN/m²) for backfill soil, the mix proportions and injection volumes of CB were determined using Equations (5) and (6), based on an assumed drilling mud density range of 1.0 to 1.5 g/cm³. The calculated results are shown in Table 5. The different mix proportions of CB I and CB II were determined according to their respective injection positions in the borehole. Due to in-situ mixing limitations, CB I accumulated in the upper borehole section may not mix adequately with drilling mud. Therefore, CB I was designed with a lower unit weight of cement and achieved a q_u of 614 kN/m² at 28-day curing period. Conversely, CB II injected from the bottom of the borehole was designed with a higher unit weight of cement, as it would be actively mixed with the drilling mud.

Table 5. Mix proportions and volumes of CB and drilling mud.

	Proportions (kg/m^3)			q_u (kN/m^2)	Volume (m^3)
	W	*C*	*B*		
CB Ⅰ	842	300	150	614	0.67
CB Ⅱ	820	400	133	-	0.70
Drilling mud	-	-	-	-	1.24

Two scenarios were considered for the mixing behavior of backfill soil in the borehole: (a) uniform mixing of CB I, CB II, and drilling mud, and (b) uniform mixing of CB II and drilling mud only. Table 6 shows the calculated mix proportions of backfill soil for these two scenarios. Figures 11 and 12 show the estimated strength ranges for both scenarios calculated using Equations (4) and (5). In both scenarios, the estimated strengths fell within the target strength (130 to 550 kN/m²) regardless of curing age. For comparison with undisturbed samples (described later), the 35-day strengths were calculated by linear interpolation between the 28-day and 56-day results.

Table 6. Mix proportions of backfill soil.

Backfill soil	Proportions (kg/m^3)			Volume (m^3)
	W	*C*	*B*	
(a) mixing scenario	768~870	184	74	2.61
(b) mixing scenario	741~896	144	48	1.94

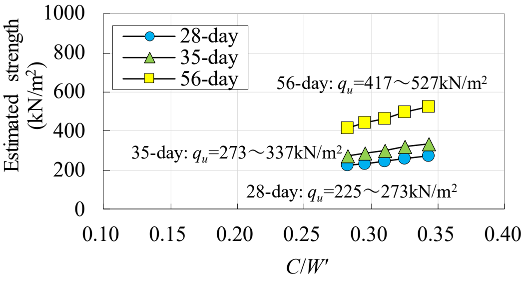


Figure 11. Estimated strength ranges of backfill soil ((a) uniform mixing of CB Ⅰ, CB Ⅱ, and drilling mud).

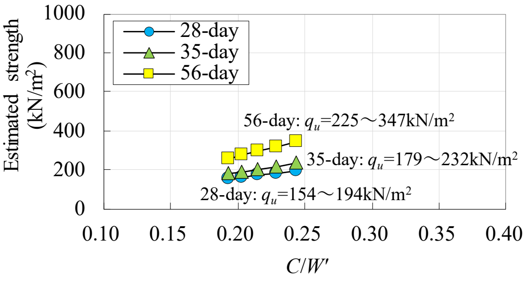
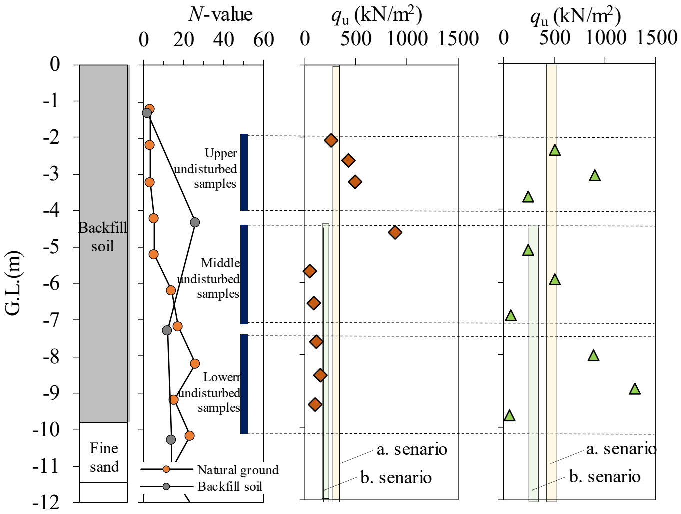


Figure 12. Estimated strength ranges of backfill soil ((b) uniform mixing of CB Ⅱ and drilling mud).

5.4 *Soil Investigation*

A standard penetration test was conducted on the backfill soil 35 days after completion of the field test. Furthermore, undisturbed samples were collected from three depths within the backfill soil (upper, middle, and lower sections) for unconfined compression tests.

Figure 13 presents the soil investigation results of *N*-value and q_u for 35-day and 59-day undisturbed samples. Figure 13a compares the *N*-value distribution of the backfill soil with that of the surrounding natural ground. The *N*-value distribution pattern of the backfill soil was similar to that of the surrounding natural ground, except at a depth of G.L.-4.3 m. At this depth, the *N*-value of the backfill soil was 26, which differed significantly from that of the natural ground. However, since the undisturbed samples at this depth contained some gravel particles, the increased *N*-value may be attributed to the effect of gravel particle contact. Figure 13b shows q_u values for the 35-day undisturbed samples and the estimated strength ranges calculated for the two mixing scenarios (a) and (b) mentioned previously. Most of the q_u values in the upper section were approximately 500 kN/m², which exceeded the upper limit of estimated strength ranges for scenario (a) and were close to the q_u of CB I (614 kN/m²). This suggests that the CB I accumulated in the upper borehole section may not have mixed effectively with the drilling mud. In contrast, the q_u values in the middle and lower sections showed relatively uniform values close to



a. *N*-value b. 35-day c. 59-day

Figure 13. Soil investigation results of field testing.

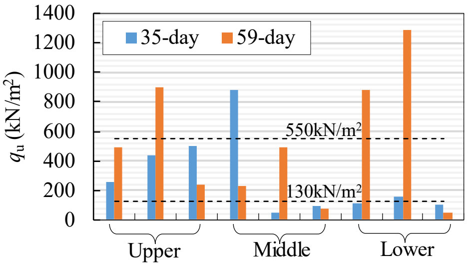


Figure 14. Comparison of target strength and undisturbed sample strength.

the lower limit of the estimated strength for scenario (b). Accordingly, it appears reasonable to conclude that CB II, injected from the bottom of the borehole, was effectively mixed with the drilling mud in the lower section of the borehole. Figure 13c shows q_u values of 59-day undisturbed samples and the 56-day estimated strength range. The q_u values of undisturbed samples exhibited large variations. One q_u value in the upper section reached approximately 900 kN/m², which may be attributed to either the extended curing period or the partial mixing of CB II with CB I.

Figure 14 shows q_u values for all undisturbed samples, along with the target strength (130 to 550 kN/m²) for the backfill soil. Despite the different curing periods, q_u values for most samples fell within or near the target strength range.

6 CONCLUSIONS

The following findings were obtained from this study:

(1) Based on the results of laboratory mix design test and unconfined compression tests, a strength estimation formula for backfill soil was proposed, that establishes the relationship between unconfined compressive strength and the effective cement-to-water ratio by considering the water retention characteristics of bentonite.

(2) A field test was conducted and the mix proportion of backfill materials were designed using the strength estimation formula. Based on the backfill soil investigation, the *N*-value distribution in backfill soil was found to be comparable to that of the surrounding natural ground.

(3) Cement bentonite slurry injected from the bottom of the borehole was effectively mixed with the drilling mud in the lower section of the borehole, and the unconfined compressive strength in this section showed relatively uniform values close to the lower limit of the estimated strength range.

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