CURRENT PRACTICE CONCERNING DRILLED SHAFTS TO SUPPORT BUILDINGS IN JAPAN

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ABSTRACT

Development of construction methods for installation of drilled shafts to support high rise buildings is being actively pursued in Japan. Of particular interest are the large diameter (about 1 to 3 meters) drilled shafts with the bottom enlarged to diameters of about 4 to 6 meters (13 to 20 feet) to provide large end bearing resistance, and drilled shafts with multi-stage enlarged nodules along the shaft in order to resist the large uplift (pull-out) force. These distinctive drilled shafts developed in recent years are found to be preferred foundation option to support high rise buildings in Japan. In this paper, we present various drilled shaft methods and types and introduce an example of a building that uses a drilled shaft with enlarged bottom in combination with multi-stage enlarged nodules.

Keywords: drilled shafts, large end bearing capacity, large pull-out resistance

INTRODUCTION

In Japan, the design practice for architecture (building structures) and civil engineering (public works) are governed by different set of design codes. As a result, the design requirements for structures related to civil engineering such as tunnels, bridges, railways, roads etc. are different from those for architecture involving privately owned building structures such as office buildings, condominiums, commercial facilities, factories hospitals etc. That is, the evaluation method for the soil bearing resistance as well as the structural strength of the pile body for a given site condition will differ in Japanese practice depending on the design code applicable to structure. For example, Table.1 shows the recommended bearing capacity of drilled shafts in sandy soils in civil engineering and architecture disciplines. As may be noted, there are different formulations recommended depending on the discipline and design standards and guidelines. In general, the end bearing capacity of drilled shafts in Japan is obtained by multiplying the bearing capacity coefficient α (60~150) by the average N value (standard penetration blow count) around the pile tip region and the base area A_p. In civil engineering discipline, only straight drilled shafts are used, but in architecture discipline, the use of drilled shafts with an enlarged bottom is permitted. While multiple piles beneath a foundation is common in civil engineering discipline, each pillar supported on single pile is often adopted in architecture discipline in view of its economic efficiency.

In this paper, we describe the use of drilled shafts in Japan in the architecture discipline. The alluvial plains where the densely populated cities are located in Japan consist of soft alluvial deposits where foundation piles with large bearing capacity are required to support building structures. In particular, for high rise buildings, drilled shaft foundations are the preferred foundation option. As a result, the development of drilled shafts with an enlarged bottom has been actively pursued in order to derive large end bearing resistance. In addition, Japan is an earthquake-prone country, and it is mandatory to design piles to resist seismic forces. Earthquake-resistant drilled shafts that can resist a large bending moment due to the inertia force of the building during an earthquake by providing a steel casing in the upper part below the pile head, and by including multi-stage enlarged nodules along the shaft to improve shaft friction to resist large uplift (pull-out) forces have been developed. This paper presents such developments in drilled shaft foundations for buildings in Japan and introduces an example of a building that uses a drilled shaft with enlarged bottom in combination with multi-stage enlarged nodules.

Table 1. Bearing capacity of the drilled shaft in Japan

Discipline		Bearing capacity (kN) = $\alpha \cdot N \cdot A_p$
Architecture	Building Standards Act	$150 \cdot N \cdot A_p$ N : Average of N values around the pile tip 1D above and 4D below the tip (D : pile tip diameter)
Architecture	Recommendations for design of building foundations	$120 \cdot N \cdot A_p$ N : Average of N values around the pile tip 1D above and 1D below the tip
Civil Engineering - Roads	Specifications for highway bridges	Sand : $110 \cdot N \cdot A_p$ ($\leqq 3300\ A_p$) Gravel : $160 \cdot N \cdot A_p$ ($\leqq 8000\ A_p$) N : Average of N values over 3D below the tip
Civil Engineering - Railways	Design standards for railway structure and commentary	Sand : $60 \cdot N \cdot A_p$ ($\leqq 3500\ A_p$) Gravel : $60 \cdot N \cdot A_p$ ($\leqq 7500\ A_p$) N : Minimum of N values over 3D below the tip

TRENDS IN DRILLED SHAFTS CONSTRUCTION METHODS IN JAPAN

The primary methods of drilled shafts construction in Japan consist of the earth drill method, the reverse circulation method, and the all-casing method in both civil engineering and architecture disciplines. Figure 1 shows the records of the three construction methods in 2021 (April 2021~ March 2022) indicating the dominance of earth drill method.

Figure 2 shows the distribution of shaft diameters and the presence or absence of an enlarged bottom based on the results of a survey conducted by the Japan Foundations Engineering Association. In civil engineering

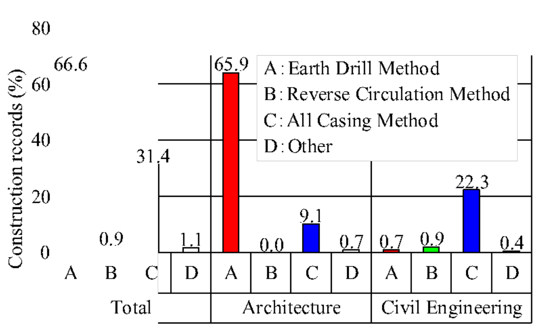


Fig. 1 Records of dilled shaft Installations

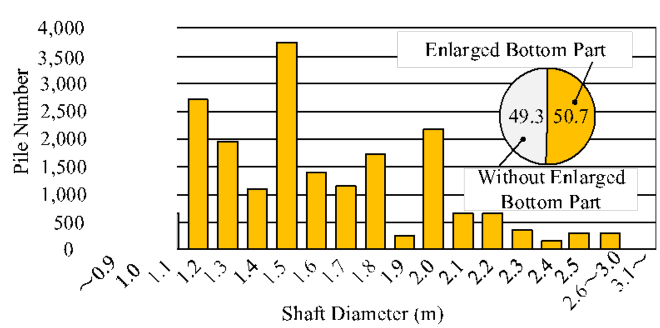


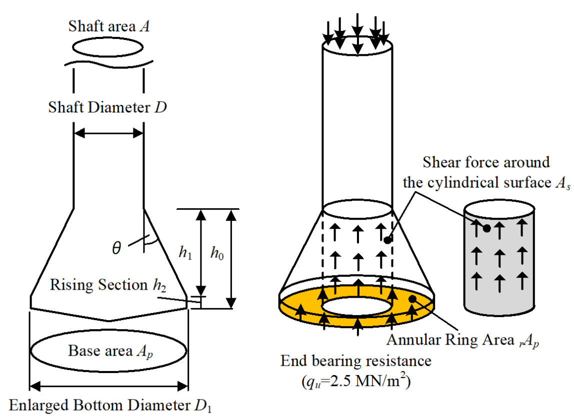
Fig. 2 Drilled shaft diameters and bottom enlargement

discipline, all-casing method is the mainstream, but in architecture discipline, the earth drill method accounts for about 70%, half of which have an enlarged bottom part. About 60% of the shaft diameters are in the range of 1.0 m ~ 1.5 m, with remaining generally in the range of 2.6 m ~ 3.0 m.

TRANSITION IN DRILLED SHAFTS WITH ENLARGED BOTTOM

In Japan, manual excavation for deep foundations started from around 1930s, and by 1960s, drilled shafts with enlarged bottom by mechanical excavation had developed. In the beginning, bottom enlargement was achieved by initially excavating a straight shaft to a predetermined depth, and using a drilling system that extends the blades by hydraulic mechanism, and then lifting the soil cuttings from the bottom by a reverse circulation method. In 1984, an enlarged bottom pile with mechanical excavation by the earth drill method was developed. The earth drill method involves two enlargement blades that are gradually expanded with the excavated soil cuttings stored in the central drum that gets lifted to ground level. Since the construction equipment for earth drill method is simpler it has mostly replaced the reverse circulation method.

In early days, the Building Center of Japan, which was conducting evaluations related to the development of drilled shafts, specified limits on shape and dimensions of enlarged bottom based on the compressive strength of concrete as illustrated in Fig. 3. Based on this, the enlarged bottom diameter was to be 4.1 m or less, the enlargement ratio (defined as the ratio of effective base area A_p to shaft cross-sectional area A) was to be 3.2 or less, the inclination angle of the enlarged bottom part was to be 12 degrees or less, and the compressive stress of concrete was between 18 and 32 MN/m^2. In addition, the upper limit of the long-term(static) allowable end bearing resistance that can be adopted was limited to 2.5 MN/m^2 assuming that it was supported by soil layer with N value of 50 or more. The rationale for limiting the inclination angle



・The diameter (D_1)of the enlarged part be 4.1 m or less.
・The base enlargement ratio (= effective base area / shaft cross-sectional area) shall be 3.2 or less.
・The inclination angle (θ) of enlarged part shall not exceed 12°.
・The length (h_2) of the rising part of the enlarged part shall not be less than 0.3 m.
・The compressive strength (F_c) of concrete shall be 18 MNm^2 ≦ F_c ≦ 32 MN/m^2.

○The Rationale of enlargement ratio 3.2

Long-term allowable bearing resistance $_aR_t = A_p \cdot q_u$
Compressive strength of shaft $_aR_p = 1/4 \cdot F_c \cdot A$
$_aR_p > {_aR_t}$
The Maximum F_c=32 MN/m^2
The Maximum D_1= 4.1m
$_aR_t = A_p \cdot q_u$ =4.1$^2 \cdot \pi/4 \cdot q_u$=13.2・2.5=33 MN
$A = 4 \cdot {_aR_t} / F_c$=4・33/32=4.125 m^2
Enlargement ratio = A_p/A =13.2/4.125 = 3.2

○The Rationale of The angle θ =12 degree

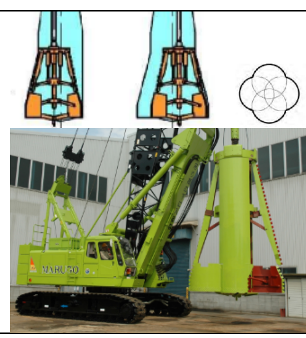
Long-term allowable end bearing resistance $_aR_g = {_rA_p} \cdot q_u$
Shear force around cylindrical surface $_aS = \tau_c \cdot A_s$
$_aS > {_aR_g}$
F_c=32 MN/m^2, h_2=1.0m, θ =12 degree
D_1= 4.1m , enlargement ratio 3.2 (D≒2.3m)
The shear stress (τ) of concrete in Japan
τ =min{ F$_c$ /40, ¾(0.49+F$_c$ /100)}=0.61MN/m^2
$_aR_g = {_rA_p} \cdot q_u$ =(4.1^2−2.3^2) π /4・q_u=9.05・2.5=22.6 MN
$_aS = \tau_c \cdot A_s = \tau_c \cdot \{(D_1-D)/2/\tan 12° + h_2\} \cdot D \cdot \pi$ =23 MN
$_aR_g / {_aS}$ = 23 / 22.6 > 1.0

Fig. 3 Specification of enlarged bottom pile and ground reaction force transmission

to 12 degrees was in consideration of the transfer of end bearing resistance from the bottom to the drilled shaft. As illustrated in Fig. 3, if the end bearing resistance (equal to 2.5 MN/m^2) acting on the annular ring area of the enlarged shaft is to be borne by the shear force around the cylindrical surface, the inclination angle needs to be limited to about 12 degrees in the case of a concrete strength of 32 MN/m^2. This was the rationale for limiting the angle of inclination to 12 degrees based on the strength of the concrete. The rationale for the enlargement ratio of 3.2 or less is also provided in Fig. 3.

However, in 2002 the limits on the shape and dimensions of enlarged bottom of drilled shafts as well as the concrete strength consideration was abolished, and drilled shafts with an enlarged bottom diameter of more than 4.1 m began to be developed. Recent technological developments in enlarged bottom piles are symbolized by the increase in the enlarged bottom diameter, the increase in the enlargement ratio by reducing the shaft diameter, and the accompanying increase in concrete strength. Table.2 is a list of the main large-diameter drilled shafts with enlarged bottom in recent years. Here, we will introduce the enlarged bottom pile (new ACE construction method) developed by four Japanese companies as a representative, which is classified as an earth drill method.

Table 2. Main large-diameter enlarged bottom piles of recent years in Japan

Method Name	Petal enlarged bottom pile	NEW-EAGLE pile	new ACE
Shaft diameter (m)	1.8～4.3	0.7～4.0	0.8～4.7
Enlarged bottom diameter (m)	3.5～6.7	0.9～5.5	0.8～4.8
Maximum enlargement ratio	−	7.29	4.62
Inclination angle	12°	21.1°	12°
Concrete strength (MN/m^2)	$18 \leqq F_c \leqq 80$	$18 \leqq F_c \leqq 60$	$24 \leqq F_c \leqq 60$
Enlarged bottom bucket			

NEW ACE CONSTRUCTION METHOD

new ACE is a construction method in which the drilling excavation corresponding to the planned shaft diameter extends to bearing after which further excavation is completed by the new ACE bucket to construct a cone-shaped enlarged part. The dimensions of the pile are shown in Table.2. The maximum enlargement ratio is 4.62 when the shaft diameter is 2.0 m and the base diameter is 4.4 m. The construction procedure is illustrated in Fig. 4.

The construction equipment consists of the new ACE excavator and the new ACE bucket as shown in Fig. 5. The new ACE excavator provides construction performance of an earth drilling method, and is equipped with features (hydraulic pump, oil transmission device, etc.) for the operation of enlargement blades. In addition, the new ACE bucket is equipped with two enlargement blades that open up or close in using the hydraulic system.

As part of the development this enlargement method, an excavation investigation was completed to confirm the excavated shape of the enlarged bottom (Fig. 5).

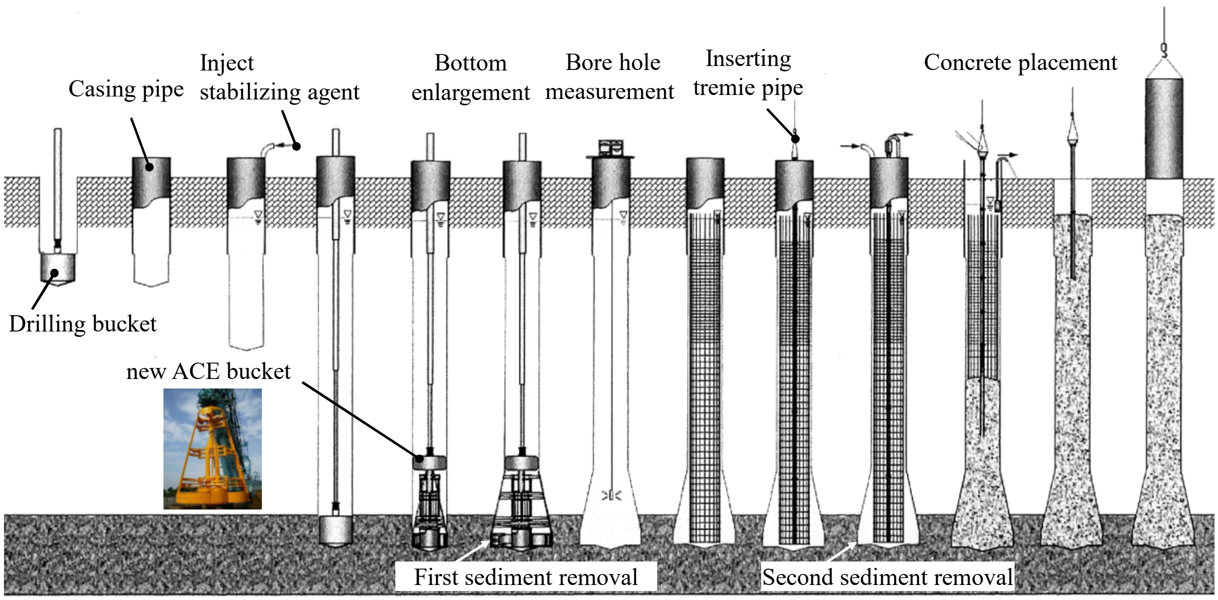


Fig. 4 Construction procedure of new ACE construction method

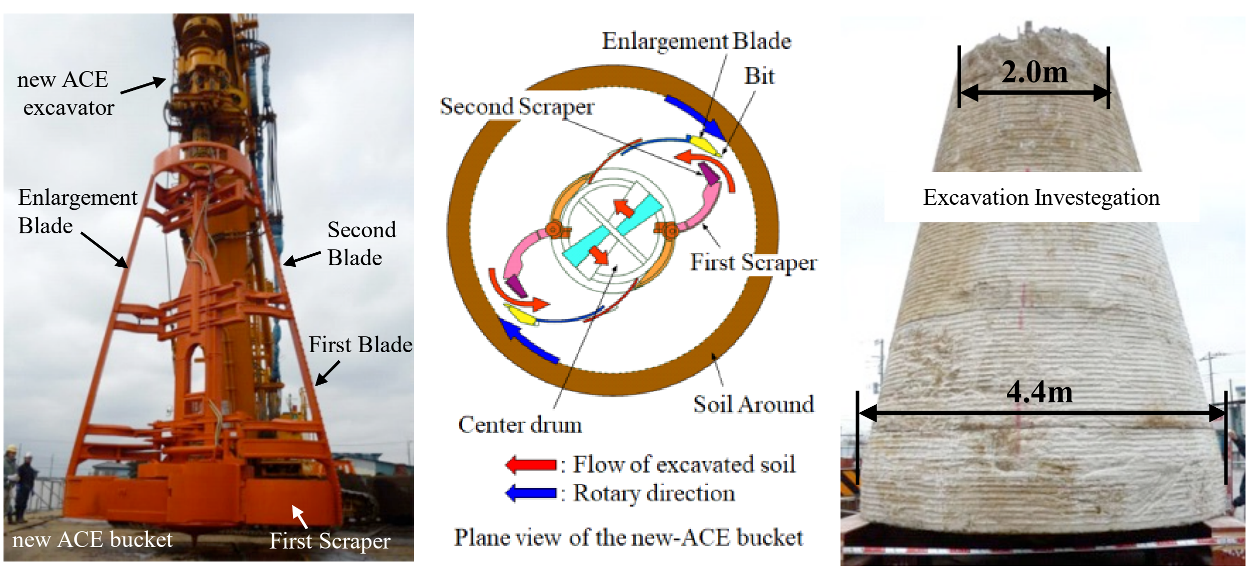


Fig. 5 Construction equipment and a result of excavation investigation

DRILLED SHAFTS WITH MULTI-STAGE-ENLARGED NODULES

Japan is well known as one of the most seismically active regions in the world. While the development of drilled shafts with enlarged bottom is primarily to resist a large compressive loads, there is also the demand for pull-out resistance under earth loading. The pull-out resistance is normally limited by available shaft friction and often results in the design of longer drilled shafts and/or larger diameters to meet the demand. Such increase in depth and/or diameter of drilled shafts start reaching practical limits as the demand increases with the need to design for higher seismic hazards prescribed by newer building codes. Faced with such circumstances, the method of evaluating the pull-out resistance of piles was revised in the Building Standards Law in 2001. With this revision, it has become possible to evaluate the unique pull-out resistance by considering the shape of the pile and the construction method. In the case of drilled shafts, a large number of drilled shafts with multi-stage-enlarged nodules have been developed in which not only the enlarged bottom part but also the enlarged nodules in the middle part are constructed to increase the shaft frictional resistance in additoin to the enlarged bottom part.

ME-A CONSTRUCTION METHOD

In this paper, we introduce the drilled shafts with multi-stage-enlarged nodules (Me-A construction method) developed by a group of nine Japanese companies. Figure 6 illustrates the general features of a drilled shaft installed by Me-A method. In this method, the drilled shaft is constructed with enlarged parts in the middle of the shaft in addition to the tip using earth drill method.The construction procedure of this method is shown in Fig. 7. The enlarged middle parts are constructed using the new ACE bucket, and then lower excavation is performed with Me ACE bucket that provides a slope to prevent sediment.

AN EXAMPLE OF A BULDING THAT USES A DRILLED SHAFT WITH ENLARGED BOTTOM IN COMBINATION WITH MULTI-STAGE ENLARGED NODULES

Outline of a building and ground overview

This is an office building with 1 basement floor and 23 floors above ground located in Japan. Considering that the facility is located directly above the station, a seismic control system has been adopted in this structure to ensure human safety and basic functions when large earthquakes occur.

Figure 8 shows a soil boring log together with drilled shaft configurations. Soft alluvium consisting mainly of clay deposits exist up to a depth of about 27 m, underlain by alternating gravel and clay. The liquefaction potential in the layer designated A_s is small, and the clay layers designated A_{c2} and D_{c2} are overconsolided. It was judged that the sand and gravel layer between depths of 28 m ~ 36 m was not suitable for bearing support layer due to inadequate layer thickness, and the sand and gravel layer at a depth of 45 m or more was adopted as the bearing stratum for the piles.

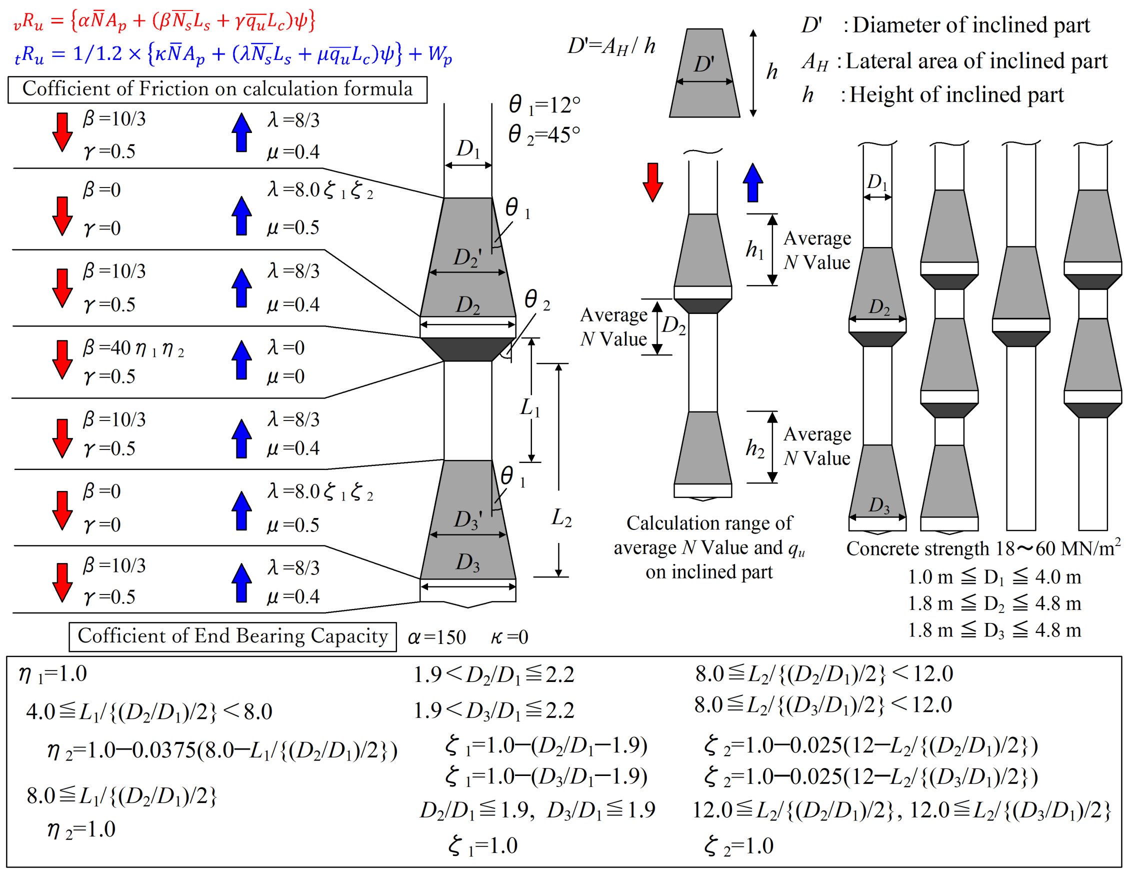


Fig. 6 Figure and specifications of Me-A

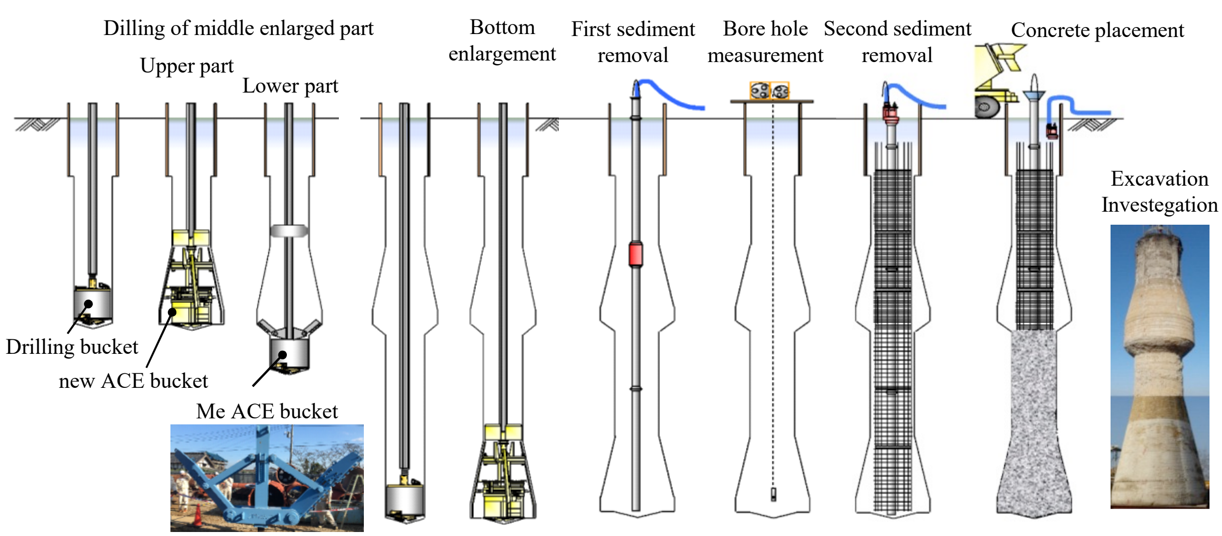
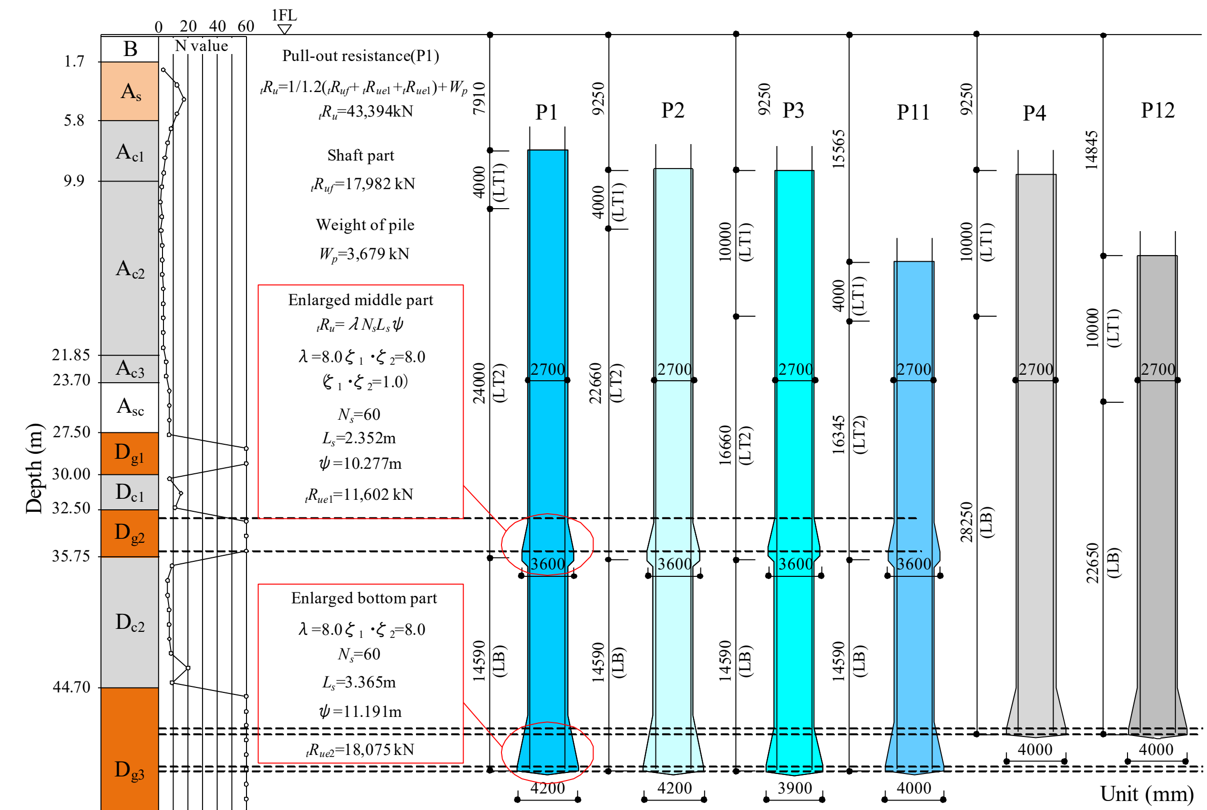


Fig. 7 Construction procedure of Me-A



		P1	P2	P3	P11	P4	P12
LT1	Main Bar	40+20 D41	40+20 D41	40+20 D41	40+12 D41	40 D41	32 D41
	Hoop Bar	D16@150	D16@150	D16@150	D16@150	D16@150	D16@150
LT2	Main Bar	40+32 D41	40+32 D41	40+12 D41	40+24 D41		
	Hoop Bar	D16@300	D16@300	D16@300	D16@300		
LB	Main Bar	32 D41	32 D41	24 D41	32 D41	24 D35	24 D35
	Hoop Bar	D16@300	D16@300	D16@300	D16@300	D16@300	D16@300
Ultimate tensile strength(kN)		41,390	41,390	34,492	29,893		

Alluvial Sandy Soil Layer	A_s	Diluvial Sandy Soil Layer	D_{s1},D_{s2}
Alluvial Cohesive Soil Layer	A_{c1},A_{c2},A_{c3}	Diluvial Cohesive Soil Layer	D_{c1},D_{c2}
Alluvial Intermediate Soil Layer	A_{sc}	Diluvial Gravelly Soil Layer	D_{g1},D_{g2},D_{g3}

Me-A Method P1 P2 P3 P11

new ACE Method P4 P12

Fig. 8 Soil boring log and pile list

Design axial load and foundation structure

A pile foundation plan is shown in Fig. 9, and ultimate axial loads are shown in Table 3. The shape of the building is 22.69m in the short side (Y direction) while the long side (X direction) is about 107 m. During an earthquake, large compressive loads (50 MN ~ 80 MN) and uplift loads (−6 MN ~ −35 MN) act on the piles in the outer periphery of the building. It is possible to resist compressive loads by using a enlaregd bottom pile (new ACE construction method), but shaft friction resistance alone is not sufficient to resist uplift loads. Therefore, we adopted a drilled shafts with multi-stage-enlarged nodules (Me-A method) and provided a enlarged part in the gravel layer at a depth of 32m~36m to provide sufficient pull-out resistance.

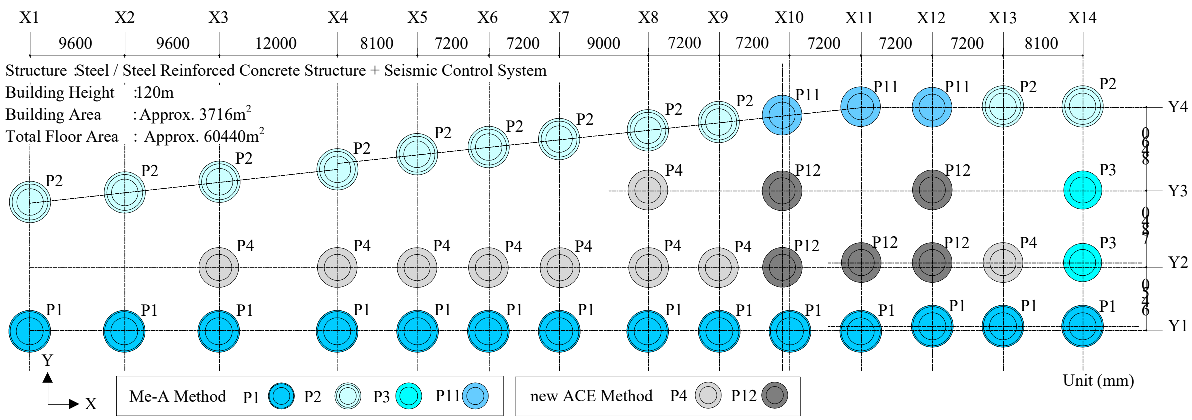


Fig. 9 Pile foundation plan

Table 3. Ultimate axial load and design values of compressive and pull-out resistance

	Pile No	Axial Load (MN)			Calculation Value (MN)			Demand to Capacity Ratio		
		N_l	N_{min}	N_{max}	${}_vR_a$	${}_tR_u$	${}_vR_u$	$N_l / {}_vR_a$	$N_{min} / {}_tR_u$	$N_{max} / {}_vR_u$
Y1-X1	P1	29.2	-27.4	83.5	34.6	-41.3	103.8	0.84	0.66	0.80
Y1-X2	P1	26.6	-25.7	75.8	34.6	-41.3	103.8	0.77	0.62	0.73
Y1-X3	P1	25.8	-30.1	77.5	34.6	-41.3	103.8	0.74	0.73	0.75
Y1-X4	P1	24.4	-33.1	75.1	34.6	-41.3	103.8	0.71	0.80	0.72
Y1-X5	P1	22.6	-30.4	68.9	34.6	-41.3	103.8	0.65	0.74	0.66
Y1-X6	P1	21.9	-27.8	66.7	34.6	-41.3	103.8	0.63	0.67	0.64
Y1-X7	P1	22.3	-26.8	67.9	34.6	-41.3	103.8	0.64	0.65	0.65
Y1-X8	P1	22.2	-30.6	69.1	34.6	-41.3	103.8	0.64	0.74	0.67
Y1-X9	P1	21.0	-31.6	67.1	34.6	-41.3	103.8	0.61	0.77	0.65
Y1-X10	P1	20.0	-30.2	66.8	34.6	-41.3	103.8	0.58	0.73	0.64
Y1-X11	P1	18.7	-28.7	61.9	34.6	-41.3	103.8	0.54	0.69	0.60
Y1-X12	P1	18.2	-27.9	63.2	34.6	-41.3	103.8	0.52	0.68	0.61
Y1-X13	P1	17.3	-31.4	59.9	34.6	-41.3	103.8	0.50	0.76	0.58
Y1-X14	P1	18.3	-34.6	68.0	34.6	-41.3	103.8	0.53	0.84	0.65
Y2-X3	P4	24.0	19.6	33.4	34.6	-41.3	103.8	0.69	-	0.32
Y2-X4	P4	22.4	12.5	34.0	34.6	-41.3	103.8	0.65	-	0.33
Y2-X5	P4	20.6	15.8	28.3	34.6	-41.3	103.8	0.59	-	0.27
Y2-X6	P4	20.2	20.2	26.7	34.6	-41.3	103.8	0.58	-	0.26
Y2-X7	P4	20.4	19.1	25.9	34.6	-41.3	103.8	0.59	-	0.25
Y2-X8	P4	18.9	18.1	23.0	34.6	-41.3	103.8	0.55	-	0.22
Y2-X9	P4	17.1	16.3	21.3	34.6	-41.3	103.8	0.49	-	0.21
Y2-X10	P12	19.7	16.6	26.9	34.6	-18.3	93.9	0.57	-	0.29
Y2-X11	P12	19.8	18.4	22.0	34.6	-18.3	93.9	0.57	-	0.23
Y2-X12	P12	20.7	15.5	25.8	34.6	-18.3	93.9	0.60	-	0.27
Y2-X13	P4	16.4	12.8	24.6	31.3	-22.9	94.2	0.52	-	0.26
Y2-X14	P3	17.3	-22.7	49.1	31.2	-34.4	89.4	0.56	0.66	0.55
Y3-X8	P4	18.4	12.9	27.0	31.3	-22.9	94.2	0.59	-	0.29
Y3-X10	P12	19.6	8.6	36.6	31.4	-18.3	93.9	0.63	-	0.39
Y3-X12	P12	20.5	10.0	37.3	29.8	-18.3	93.9	0.69	-	0.40
Y3-X14	P3	18.9	-6.4	46.5	31.4	-34.4	89.4	0.60	0.19	0.52
Y4-X1	P2	28.8	-25.5	83.0	31.3	-41.3	103.8	0.92	0.62	0.80
Y4-X2	P2	26.8	-25.3	75.5	31.3	-41.3	103.8	0.86	0.61	0.73
Y4-X3	P2	26.8	-21.0	74.8	29.8	-41.3	103.8	0.90	0.51	0.72
Y4-X4	P2	24.4	-24.1	71.1	34.6	-41.3	103.8	0.70	0.58	0.68
Y4-X5	P2	23.6	-24.8	68.5	34.6	-41.3	103.8	0.68	0.60	0.66
Y4-X6	P2	23.2	-20.4	66.6	34.6	-41.3	103.8	0.67	0.49	0.64
Y4-X7	P2	21.1	-24.9	62.5	34.6	-41.3	103.8	0.61	0.60	0.60
Y4-X8	P2	19.9	-23.8	59.2	34.6	-41.3	103.8	0.57	0.58	0.57
Y4-X9	P2	23.2	-25.4	71.5	34.6	-41.3	103.8	0.67	0.61	0.69
Y4-X10	P11	22.4	-21.7	65.0	31.4	-29.8	94.2	0.71	0.73	0.69
Y4-X11	P11	23.4	-22.2	69.2	31.4	-29.8	94.2	0.75	0.74	0.73
Y4-X12	P11	23.4	-22.2	69.2	31.4	-29.8	94.2	0.75	0.74	0.73
Y4-X13	P2	22.7	-23.8	64.8	34.6	-41.3	103.8	0.66	0.58	0.62
Y4-X14	P2	22.9	-25.5	68.2	34.6	-41.3	103.8	0.66	0.62	0.66

※${}_vR_a$=1/3 ${}_vR_u$ N_l : Long-term axial load , N_{min} : Axial ultimate load (tension) , N_{max} : Axial ultimate load (compression)

Evaluation of the bearing capacity of the Me-A method

The ultimate compressive bearing capacity ${}_vR_u$ and the ultimate pull-out resistance ${}_tR_u$ determined from the ground resistance of the drilled shaft by Me-A method are obtained using the calculation formula shown in equations [1] and [2] respectively. For evaluation of the end bearing capacity and the resistance of the enlarged middle part, consideration is given to the influence of (1) the diameter enlargement ratio and (2) the enlargement interval ratio, by allowing for the resistance of the inclined part in the enlarged parts in formulating the friction resistance. As for the compressive bearing capacity, there is no effect of the enlargement ratio, and the effect of the enlargement interval ratio is considered. From the experimental results, it was confirmed that the bearing capacity tended to decrease when the enlargement interval ratio fell below 8.0, so the reduction factor η_2 was set as shown in Fig. 6. As for the pull-out resistance, since both effects were confirmed from the experimental results, the reduction coefficients ζ_1 and ζ_2 were set. Each cofficient and reduction factor is shown in Fig. 6.

$$_{v}R_u = \left\{\alpha \overline{N} A_p + (\beta \overline{N_s} L_s + \gamma \overline{q_u} L_c)\psi\right\} \quad [1]$$

$$_{t}R_u = 1/1.2 \times \left\{\kappa \overline{N} A_p + (\lambda \overline{N_s} L_s + \mu \overline{q_u} L_c)\psi\right\} + W_p \quad [2]$$

Where α, κ = cofficient of end bearing capacity, $\overline{N}$ = average of N value around the pile tip (Blow 1D, Upward 4D from the tip (D : pile tip diameter)), A_p= effective area of pile tip, β, λ =cofficient of friction of sand, $\overline{N_s}$ = average of N value of sandy ground around the pile, L_s = total length on sandy ground, γ, μ = cofficient of friction of clay, $\overline{q_u}$ = average of compressive strength of clay ground around the pile, L_c = total length on clay ground, ψ = length of the circumference of pile, W_p = effective dead weight of pile.

Result of compressive bearing capacity and pull-out resistance

A list of compressive bearing capacity and pull-out resistance capacity are shown in Table 3. The adopted calculation values is the smaller of the compressive bearing capacity or pull-out resistance determined from the ground resistance and the compressive strength or tensile strength of the pile cross-section. In addition, the demand to capacity ratio of the axial force of each column position to the adopted bearing capacity is shown in Table 3. By adopting the Me-A, we were able to keep pulling within 0.85 of the demand to capacity ratio.

CONCLUSION

The latest drilled shafts in Japan were explained. An example was shown in which it was difficult to design economically without adopting drilled shaft with enlarged bottom in combination with multi-stage enlarged nodules.

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