of these states use both mostly depending on the ground types. For a larger hole, twenty-three indicate with their specific guidelines in terms of the size of the hole varying from slightly larger than the size of pile to 150 mm (6 in) or larger. Eight states use a 150 mm (6 in) or larger hole. Five states use a 50 mm (2 in) larger hole and four states use a slightly larger hole. Three states use a 100 mm (4 in) larger hole, and three more states use a 25 mm (1 in) larger hole. Five other states do not provide a specific hole size, and two other states suggest the hole size to be larger but related to the size or shape of the pile. Seven states indicated they only use a smaller hole, while 11 other states use either smaller or larger depending on the ground types. Fifteen states either do not have a published guideline or did not provide the info. The distribution of the pilot hole adoption in terms of its size is provided in Figure 4.

The size of the pilot hole is another factor affecting the design because the length and diameter of the hole affect the development of the shaft resistance of the pile. Information in regard to the size of the pilot hole came mostly from the review of agencies standard specifications, while the information on how skin friction is respectively considered due to the size of the pilot hole and the length mostly came from answers of the survey sent out to state agencies.

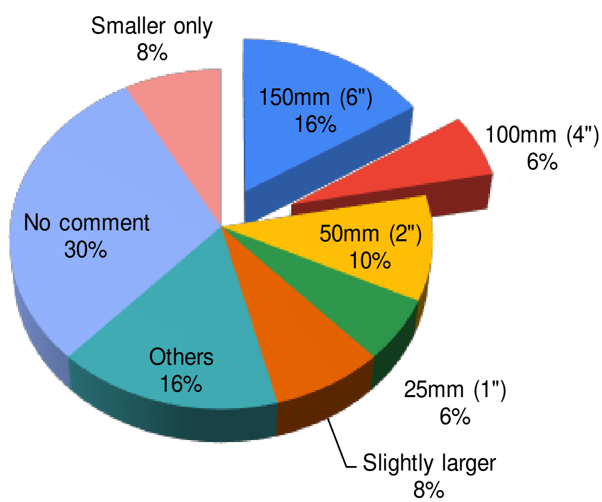

Figure 4. Size of the pilot hole used

A few states specifically define when to consider the skin friction depending on the socketing of the pile into the rock layer, the depth of the penetrated hard layer, or the length of the pilot hole. However, in most cases, the skin friction is neglected when the diameter of the holes is larger than that of the pile, while it is considered when the hole is smaller than the pile size. Excluding 9 states that did not provide the information on the skin friction in the pile with a pilot hole design, thirty-four states ignore the skin friction especially when the size of the hole is larger, or when it is installed on rock. One state considers the skin friction per designer's choice while the other state considers it only if it is backfilled with concrete. Only five states indicated they consider the skin friction. On the other hand, two states seem to be more conservative as the skin friction is not considered even if the hole size is smaller. The distribution of states' consideration on the skin friction is presented in Figure 5.

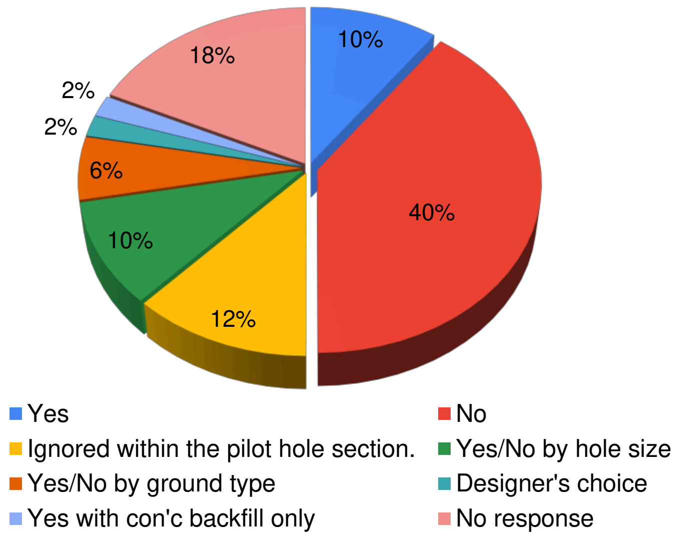

Figure 5. Consideration of skin friction in the pile design with a pilot hole

When the pilot hole is larger than the pile, the void space around the pile is supposed to be backfilled but different materials were reported to be used. Common materials used for the backfill are sand, gravel, or concrete.

Pile Installation

According to the survey results, even though a pilot hole is used in most states, the purposes of using it are different, and consequently, the design and installation methods are also different. Some of the identified reasons of using the pilot holes are i) to reach the required tip elevation thru resistant strata without being damaged or to increase lateral stability, ii) to reduce downdrag loading on the pile, iii) to protect adjacent structures from vibration, and iv) to achieve simple and cost effective construction when a hard rock layer exists.

The first three reasons are usually encountered with a pilot hole in soil. In most cases with these reasons, the piles are driven to the designated tip elevation with a larger or smaller pilot hole, and the pile driving analyzer (PDA) is usually utilized for actual driving refusal or capacity verification. On the other hand, most states did not specifically indicate if the driving is required or not when the pilot hole is used for the last reason that driving is not necessarily required to seat the pile on the hard rock layer. This approach has advantages in simple construction procedure and its cost. However, there are no established design guidelines for this case, and thus, the current designs and construction methods seem to be largely based on experience and engineers' judgement. In addition, there is yet no appropriate method to verify the capacity of the pile. A couple of states have indicated they design such a pile as a drilled shift when used with a rock socket.

CONCLUSIONS & DISCUSSION

Drilling a hole before driving a pile seems to be a common practice that many state agencies have been using either regularly or occasionally when dense soil or rock layers are confronted in the middle of or at the bearing layer. The current study reviewed each state agency's terminology and suggestions in their design and construction approaches including the hole size, backfill method, ground conditions, and consideration of skin friction. Of the reviewed states there were some noticeable commonalities and differences.

Several different terms are being used for the same method, which may cause confusion. Some states use different terms for the installation in soil and rock, while some states use one term for both. This study suggests the use of one of the terms, pilot hole, because it clarifies the role of the hole as a pile driving assistant method and also prevents confusions with similar terms being used for other deep foundation types.

It is also found that the pilot hole as a pile driving assistant method is used for both soils and rocks, and the size of the hole is suggested to be larger in general ignoring the skin friction in most states. However, some states suggest the smaller pilot hole for soil while taking into account the skin friction. A few states consider the skin friction but only ignoring it within the pre-bored section. For rocks, it seems to be common to consider the end bearing capacity only or even limit the pile capacity to the structural capacity of the pile. Most states still consider it as a driven pile with some special considerations for the skin friction, whereas one state treats it as a drilled shaft when designing the pile.

The use of pilot holes brings some advantages such as reducing the vibration, noise, and pile damage during the installation, while it also reduces the skin friction regardless of the hole size and eventually the capacity of the pile. However, there is no federal level general guidelines on the use of the technique yet especially with the LRFD method when a pilot hole is adopted. It is being considered differently in different states, which may lead to inaccurate estimation of the capacity. Therefore, additional study is necessary to develop a common design methodology and parameters for this type of pile under the new LRFD guidelines.

ACKNOWLEDGEMENT

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Increased Axial Resistance of Small-Diameter Piles and Ground Anchors by Using Expander Bodies

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ABSTRACT

An expander body (EB) is an element consisting of a folded steel sheet with a circular cross section that can be added to a relatively small diameter pile and ground anchor to increase axial resistance. During the injection of pressurized grout, the outer portion of the EB is expanded (inflated) from its original diameter of about 120 mm (4.75 in.) to about 400–800 mm (15.75–31.5 in.), depending on the model and stiffness of the in situ soil surrounding the device. Due to the controlled expansion process, the stress state is improved around the device and can be deduced from the recorded measurements; hence, the available axial resistance for these smaller diameter elements can be increased considerably due to the greater soil stiffness and shear strength around the device as well as to the larger cross-sectional area (i.e., end resistance) and larger surface area. This paper describes the general principles of the EB system as well as the construction, grouting operation, quality control, and select results from static axial compression load tests performed on continuous flight auger (CFA) + EB piles.

Keywords: Expander body, Pressuremeter test, Quality control, Axial resistance, Post-grouting

INTRODUCTION

The expander body (EB) system has been used successfully in a wide range of ground conditions from very loose sands/silts to very dense sands and medium-dense gravels as well as in soft to stiff clays. This technique has been used successfully with different pile types and ground anchors for a variety of applications, including for support of high-rise buildings, bridge structures, and industrial facilities as well as in conjunction with support of excavation and underpinning works (Sellgren et al, 1989; Terceros et al, 1995; and Fellenius and Terceros, 2014). The EB system is added to the lower portion of various smaller diameter piles (e.g., continuous flight auger (CFA) piles, augercast-in-place (ACIP) piles, micropiles, and drilled displacement (DD) piles) and ground anchors to increase the axial resistance through the improvement of the axial side and/or end resistance. When an EB is used, the design is essentially separated into two parts – design of the conventional pile/anchor element (above the EB) and of the EB system. Since the initial cross-section of the EB is relatively small, conventional equipment used to install the piling or ground anchor can be used to create the borehole for the pile/anchor+EB. The effects of the expansion of the EB results in a compaction of the soil and an increase in both the resistance at the soil-EB interface and the stiffness of the soil up to a distance of 3 to 4 diameters from the center of the element. The increase in stiffness is directly correlated to the initial stiffness and stress state of the soil and to the final expanded diameter of the EB. Based on test results on various projects, an pile/anchor+EB has demonstrated that the required resistance can be achieved, in general, within a shorter element length.

OVERVIEW AND BASIC PRINCIPLES

The following discussion describes the principles and process for an axially compression loaded EB-pile system, but similar principles apply for ground anchors axially loaded in tension. An EB system consists of essentially three integrated components: a shell with a circular cross section (i.e., a folded steel sheet made into a tube, Figure 1), a central reinforcement section, and primary and secondary grouting mechanisms. The primary grouting mechanism is used to expand the shell while the secondary mechanism is used to perform post-grouting below the pile toe. The relative expansion of the EB and the associated response (i.e., shape of the grout volume vs. injection pressure relationship) is a function of the model used and the geotechnical conditions of the in-situ soil (e.g., in-situ stress state, strength, and stiffness).

A pile+EB assembly (e.g., reinforcement steel/sections, EB attachment, and instrumentation) is inserted into the grouted/concreted borehole, which was formed using the applicable installation technique. After the grout/concrete has set, primary grouting can be performed at any time irrespective of the grout/concrete strength (i.e., the layer of grout/concrete surrounding the EB is thin and can be easily fractured as tension stresses are induced during expansion). During primary grout injection (Figure 1), the shell undergoes two distinct responses: (1) expansion outward into the surrounding soils and (2) a reduction in the effective length (i.e., shortening) of the EB (Table 1). The diameter of the EB prior to injection is about 120 mm (4.7 in) and can expand up to a diameter ranging from 400 to 800 mm (15.7 to 31.5 in). Based on the volume of grout injected, the effective diameter of the EB can be determined using the appropriate calibration curve for the particular model used. The incorporation of post-grouting below the pile toe is intended to address potential soil decompression due to the shortening of the element and to ensure contact between the pile toe and in-situ soil (this discussion is beyond the scope of this paper).

(a) Pre-expansion (b) During the various stages of expansion (c) Fully expanded

Figure 1. Representation of the different stages of expansion (model EBI612 shown)

During expansion, the soil surrounding the EB will undergo changes to its stress state (e.g., change in void ratio) as a function of the soil type, original stress state, and consistency, whereby any ground modification should result in larger values of unit side resistance. The pressure required to initiate expansion is equal to the horizontal stress acting on the EB from the surrounding soil. During the initial phases of primary grouting, disturbed soil around the device is recompressed. Then, as primary grouting is continued, the in-situ soil is compressed as the shell expands. Based on experience, general trends have been realized in the grout volume injected-vs.-injection pressure relationship (i.e., stress-strain response) for different ground conditions and piling installation methods, which facilitates the deduction of the local soil type and engineering properties (Marinucci et al, 2020). Estimates of the axial resistance can be used to confirm the adequacy of the design or facilitate modifications to the design (e.g., increase in pile length) using the recorded measurements without waiting for a full-scale load test to be performed. In general, the greater the pressure that can be applied during grouting, the greater the axial resistance of the completed element.

Table 1. Details of select EB models

EB Type	Pre-expansion		Post-expansion (fully expanded)				
	Diameter D_{EB} (mm) [in]	Length L_{EB} (m) [ft]	Diameter D_{EB} (mm) [in]	Length L_{EB} (m) [ft]	Area at Toe $A_{EB,b}$ (sq m) [sq ft]	Side Surface Area $A_{EB,s}$ (sq m) [sq ft]	Volume V_{EB} (liter) [cu ft]
EBI412	120 [4.7]	1.2 [3.9]	400 [15.7]	0.92 [3.0]	0.13 [1.4]	1.16 [12.4]	110 [3.9]
EBI615	125 [4.9]	1.5 [4.9]	600 [23.6]	1.26 [4.1]	0.28 [3.0]	2.83 [30.5]	360 [12.7]
EBI820	145 [5.7]	2.0 [6.6]	800 [31.5]	1.76 [5.8]	0.50 [5.4]	4.42 [47.6]	880 [31.1]

In cohesionless soils, the compactive effort results in a decrease of the void ratio (higher relative density than initial) relatively soon after expansion is completed. For cohesive soils, the soil will be deformed plastically relative to its consistency and may require some time to realize the increase in shear strength as the soil undergoes consolidation within the affected zone. In partially saturated and fully saturated soils, the expansion may generate excess pore water pressures in the soil surrounding the EB. For cohesionless soils with minor fines content (<15%), the dissipation of these induced excess pore water pressures should be relatively rapid. For cohesionless soils with appreciable fines (>15%) and for cohesive soils, however, the dissipation of the induced excess pore water pressures will require time to dissipate, which will depend on the length of the drainage path and the hydraulic conductivity of the soil.

For an EB-pile system, the design is essentially separated into two components, as shown in Equation 1: (1) estimation of side and lateral resistances provided by the pile (using conventional design methodologies) above the EB and (2) estimation of side and end resistances provided by the EB. For the EB, the total axial resistance in compression is estimated using Allowable Stress Design (ASD). The nominal resistance is based on cavity expansion theory as described in the recommendations in LCPC-SETRA (1985). During the design phase, the axial resistance contributed by the EB (Equations 2 and 3) is based on the base and side surface areas assuming the EB will become fully expanded (provided by manufacturer) and is determined using empirical correlations for the engineering parameters obtained during the site characterization program. However, the as-constructed axial end and side resistance are computed using Equations 4 and 5, which use the effective base and side surface areas determined using the measurements of grout volume and injection pressures during expansion. Additional details about the design process, real-time monitoring, and quality control are presented in Marinucci et al (2020).

$$Q_{ta,EB} = \frac{Q_{e,EB,i}}{FS_e} + \frac{Q_{s,EB,i}}{FS_s} \quad (1)$$

$$Q_{e,EB,d} = \{k[2p_{lim,d} - \sigma_h] + \sigma_v\}A_{e,EB,d} \quad (2)$$

$$Q_{s,EB,d} = p_{lim,d}A_{s,EB,d}tan\delta \quad (3)$$

$$Q_{e,EB,f} = kp_{lim,EB}A_{e,EB,f} \quad (4)$$

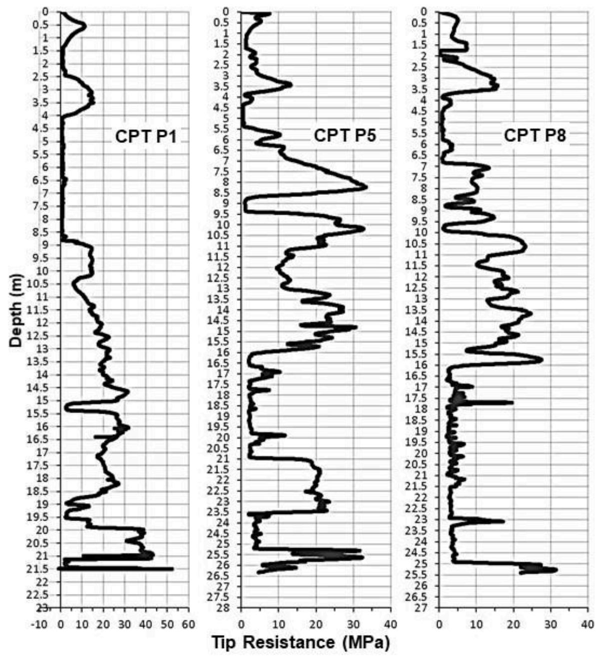
$$Q_{s,EB,f} = p_{lim,EB}A_{s,EB,f}tan\delta \quad (5)$$

where: $Q_{ta,EB}$ = Total allowable axial resistance provided by the EB
$Q_{e,EB}$ = Nominal, unfactored axial end resistance provided by the EB
$Q_{s,EB}$ = Nominal, unfactored axial side resistance provided by the EB
i = Design phase (d) or post-expansion phase (f)
FS_e = Factor of Safety for end resistance (e.g., 3.0)
FS_s = Factor of Safety side end resistance (e.g., 2.5)
k = Empirical bearing capacity factor (Clarke, 1995)
$p_{lim,d}$ = Limit pressure based on in-situ measurements (pressuremeter (PMT) p_{lim}, standard penetration test (SPT) N_{60}-values, cone penetration test (CPT) q_t and f_s, etc.)
σ_h = Total horizontal stress acting at the base of the EB
σ_v = Total vertical stress acting at the base of the EB
$A_{e,EB,d}$ = Area of the base of the (assumed) fully expanded EB
$A_{s,EB,d}$ = Side surface area of the (assumed) fully expanded EB
δ = Friction angle between dissimilar materials = 10 degrees
$p_{lim,EB}$ = Limit pressure based on grouting measurements (i.e., pressure vs. injected volume)
$A_{e,EB,d}$ = Area of base of expanded EB (calibration curve and injected grout volume)
$A_{s,EB,d}$ = Side surface area of expanded EB (calibration curve and injected grout volume)

MINI CASE HISTORY

When completed, the $US50 million Manzano 40 Plaza Empresarial building development will consist of two 30 story towers, each with 800 parking spaces, multiple restaurants, and 260 offices. One of the towers will also house an executive hotel. In addition, this building project is the first in Bolivia to incorporate BIM (Building Information Modeling) and has integrated and coordinated more than 20 simultaneous engineering projects via the 3D technology. The overall footprint of the development is approximately 128 m (420 ft) by 57 m (187 ft). The site characterization program was conducted in two parts and consisted of performing 3 soil borings and standard penetration testing (SPT), 7 cone penetration testing (CPTu), 17 pressuremeter tests (PMTs), 2 Marchetti flat dilatometer tests (DMT), surface wave geophysical testing, and one-dimensional consolidation testing. The soil borings were performed to a depth of about 40 m (131 ft) and the CPT soundings were pushed to a depth ranging from 21 to 27 m (69 to 89 ft).

Select profiles of CPT tip stress (performed by others) are presented in Figure 2 and the associated subsurface soil profiles with deduced soil behavior types are presented in Figure 3. At the time the soil borings were performed, the groundwater table was found to be at a depth of about 4.5 m (14.8 ft). The clays encountered while performing the soil borings were characterized as normally consolidated, which were later verified during the laboratory consolidation testing. Based on the engineering properties of the in-situ soils, anticipated loading from the superstructure, and possible differential settlements, the design engineer recommended using pile foundations beneath the more heavily loaded areas (e.g., support of column footings, elevators, and shear walls) and shallow strip footings for support of the lightly loaded structural elements and for the perimeter earth retaining walls.



CPT-P1 = west side of west tower
CPT-P5 = near center of site
CPT-P8 = east side of east tower

Figure 2. Select profiles of CPT tip stress

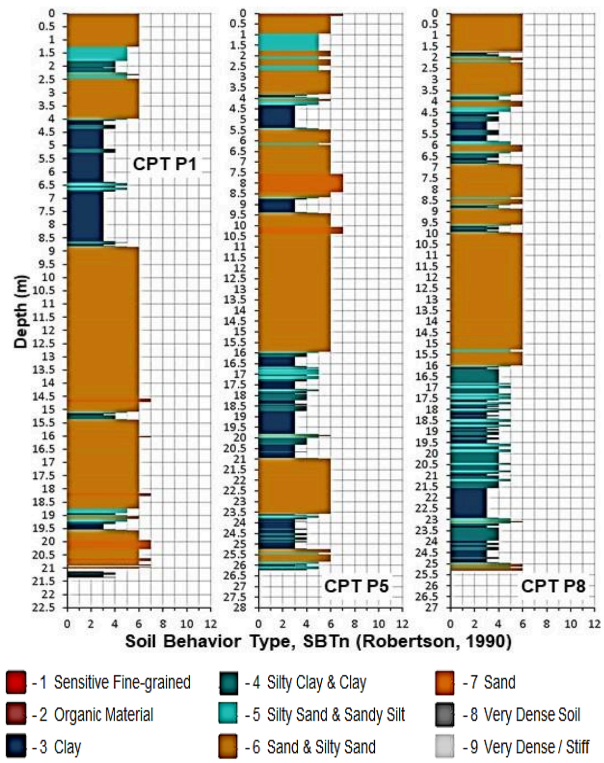


Figure 3. Subsurface profiles of soil layers

The details of the original design (layout not shown) are provided in Table 2. Based on a value engineering design and a technical-cost benefit analysis, the foundation contractor proposed an alternative foundation system consisting of a combination of two piling types: (1) large diameter full displacement piles (FDPs) to support the lighter loaded areas (replacing the 800 mm drilled shafts) and (2) CFA+EB piles to provide support in the more heavily loaded areas of the two towers (replacing the 1200 mm and 1500 mm drilled shafts). The overarching concept for the alternative was to optimize the foundation system (e.g., quantity and geometry) while providing more homogenous settlements across the structures and towers. Based on conversations with and analysis by the structural engineer, the layout of the piles (Figure 4) was revised and the axial compressive design load for each FDP and CFA+EB pile was revised to be 55 tonne (60 ton) and 260 tonne (287 ton), respectively.

The geotechnical designs for the FDPs and for the length of the CFA pile above the EB were performed using an approach similar to that discussed by Nesmith (2002) and Brettmann and NeSmith (2005). Adjustments to the design approach were made to account for local soil conditions, which were based on the contractor's experience in the region, recommendations by

Fellenius (2018), and on the guidelines in Section 7.6, Part 1 of Eurocode 7 (EN 1997-1, 2004). The geotechnical design for the EB below the CFA pile was performed using Equations 2 and 3.

Table 2. Details of the original deep foundation system

Pile Type	Quantity	Diameter mm (in)	Embedment Length m (ft)
Straight-sided drilled shaft (with slurry)	110	800 (31.5)	10.5 (34.5)
Straight-sided drilled shaft (with slurry)	40	1200 (47)	28.5 (93.5)
Straight-sided drilled shaft (with slurry)	135	1500 (59)	28.5 (93.5)

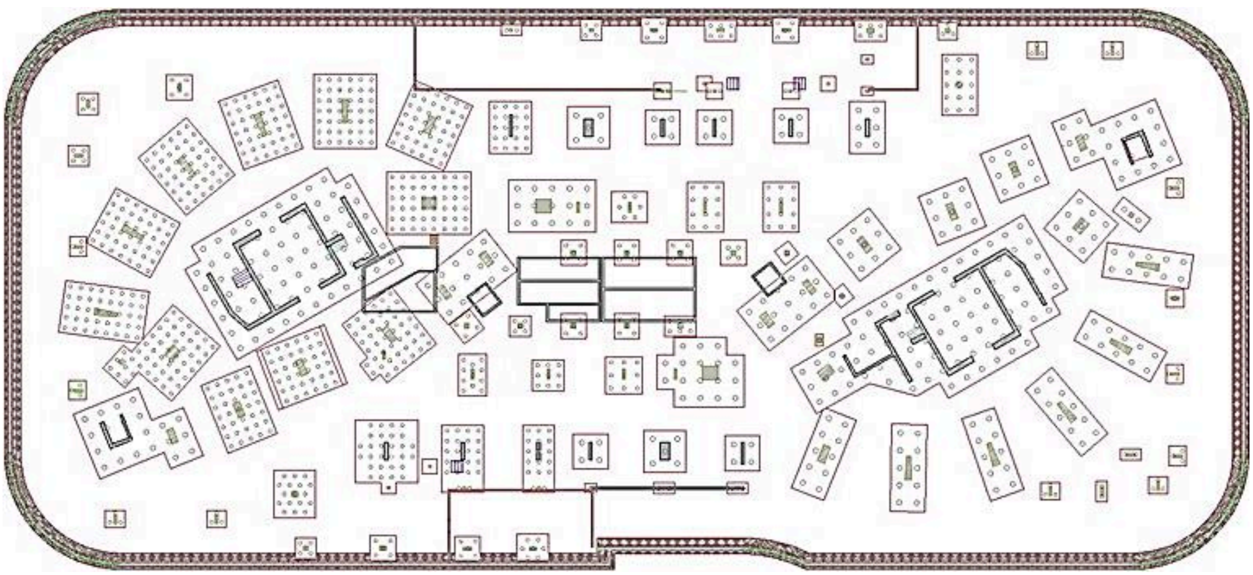


Figure 4. Plan view of the contractor-designed foundation alternative

Following the geotechnical design, the commercial software programs *UniPile* and *UniSettle* were used to perform the structural and settlement analyses, respectively, and determine the load-displacement behaviors of the individual piles and the pile groups. The final foundation layout consisted of (a) 804 each, 360 mm (14.2 in) diameter FDPs, each with an axial design capacity of 55 tonne (60 ton) and tipped at Elev. -11.0 m (Elev. -36.1 ft) and (b) 406 each, 500 mm (19.7 in) diameter CFA+EB-812 piles, each with an axial design capacity of 260 tonne (287 ton) and tipped at Elev. -28.0 m (Elev. -91.9 ft). For the project, the maximum allowable differential settlement between columns was L/400. Based on the analyses, a maximum vertical settlement of about 67 mm (2.6 in) was estimated at the center of each tower under full application of the applied loading, and the predicted differential settlements were within the tolerable limits. Details of the CFA+EB piles are provided in Figure 5.

Per the project requirements, three production CFA+EB piles were to be tested in axial compression to confirm the axial resistance provided by the piles and load-displacement response at the top of the pile. However, evaluation of the load transfer behavior along the length of the pile was not requested as part of the testing, hence strain gauges were not installed within the piling elements. Pertinent data from the expansion of the EBs for the three test piles are