Complexities of Mixed Foundation Systems for Boston Highrise

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Abstract

This case history discusses geotechnical design challenges of constructing an approximately 146 m (480 ft) tall high-rise building in the heart of Boston, abutting an existing structure (parking garage and office building) that will remain in service during construction. Interactions between Haley & Aldrich, the Geotechnical Engineer, and McNamara | Salvia, the Structural Engineer, were paramount in the foundation and structural design. Due to the loading conditions and various physical site constraints, foundations consist of high-capacity drilled-in micropiles, drilled shafts, and load-bearing elements. Geo-structural parameters such as foundation vertical and lateral stiffnesses were critical in modeling the structure behavior and were relied upon to resolve loading down through foundation elements, which was complicated by the constrained site configuration and dimensions. The iterative nature of this project's building design highlights the importance of interaction between the geotechnical and structural disciplines and the technical expertise required to evaluate complex soil-structure interaction.

Introduction

The Bulfinch Crossing development is a multi-phased, mixed-use development project consisting of three new high-rise towers and three mid-rise structures in the Government Center district of Boston, Massachusetts, orchestrated by National Real Estate Advisors and the HYM Investment Group, LLC. The first phase to be constructed is an approximately 146 m (480 ft) tall Residential Tower which is being built abutting an existing and operational parking garage to remain in place. A significant portion of the existing parking garage will be reused and incorporated into the final buildout and will remain in service during construction. The overall planned development is depicted in Figure 1, courtesy of the HYM Investment Group, LLC.

Figure 1. Bulfinch Crossing Development overview rendering.

Background

The overall development is currently an eleven-story precast concrete structure with partial below-grade space ("Government Center Garage," or "GCG"). GCG is comprised of a nine-level operational parking garage with the top two levels consisting of office space. The structure is supported on concrete-filled steel pipe piles bearing in glacial till deposits or weathered bedrock that underlie the site. GCG is bounded by John Fitzgerald Surface Road to the east and New Chardon, Bowker, and Sudbury Streets to the north, west, and south, respectively. The garage ground floor level is divided by Congress Street, where the upper parking levels span above. The eastern end of the garage provides cover for the MBTA Haymarket Subway Station headhouse and the MBTA Haymarket Bus Station.

Government Center Garage was originally constructed in 1967 with the structure footprint located on the edge of a former colonial-era Mill Pond that was filled in the mid- to late- 1800s and later occupied by surface streets and buildings, as shown in Figure 2.

Figure 2. 1775 and 1890 overlays of GCG over colonial pond and pre-garage city grid.

Proposed development

The approximately 146 m (480 ft) tall high-rise, currently referred to as the Residential Tower at Bulfinch Crossing, is planned to be a 46-story structure with one basement level. The tower footprint is situated within the southwest corner of the site, where a section of the parking garage previously existed, bound by Bowker Street to the west and Sudbury Street to the south as shown in Figure 3.

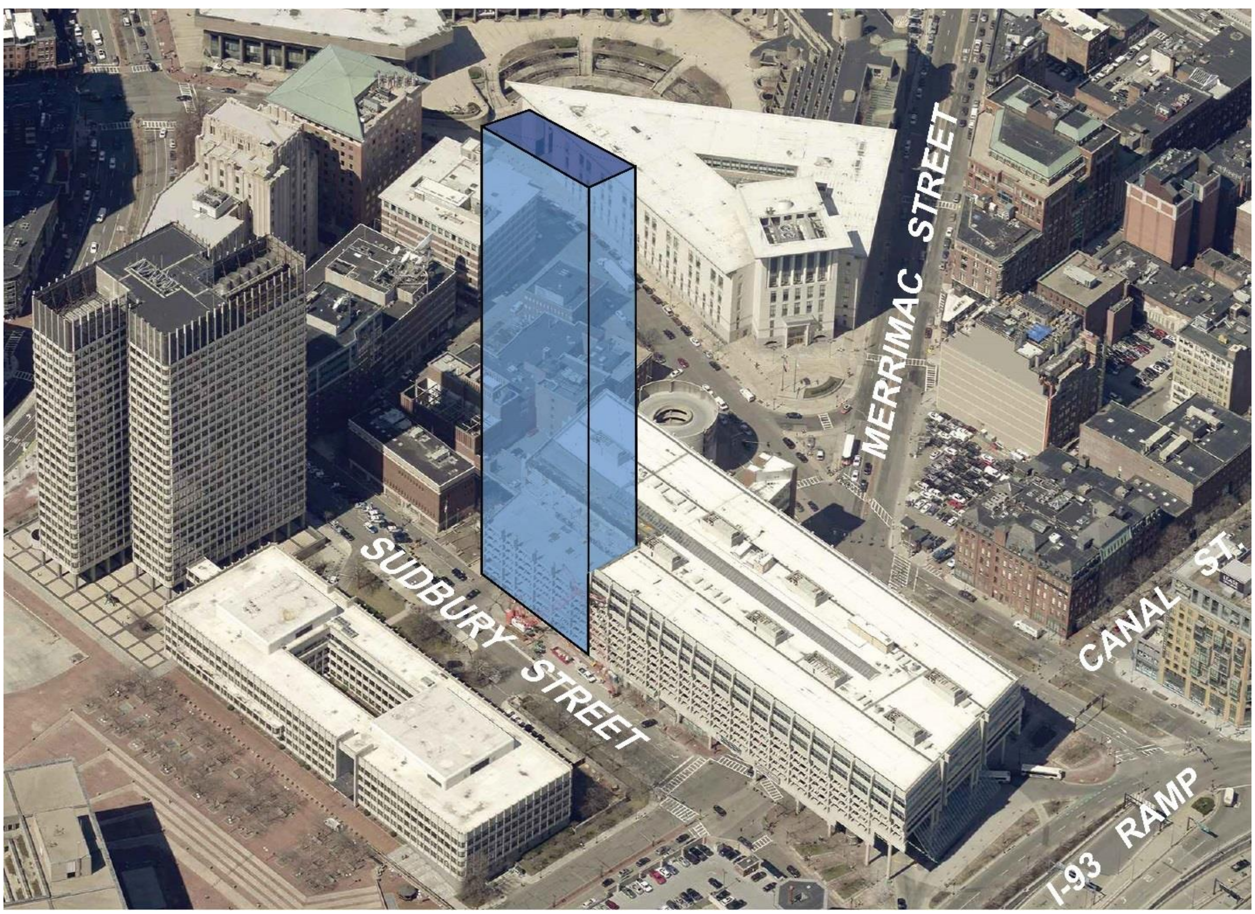

Figure 3. Proposed tower footprint.

Before tower construction could begin, enabling work was required to re-route parking garage traffic, maintain ongoing garage operations, and allow for demolition of the southwestern corner of the garage structure where the tower was to be located. The enabling work included construction of at- and below-grade parking ramps as well as a brace frame to support a rearranged structure and parking space layout and a future roof deck.

Subsurface conditions

Site stratigraphy generally consists of the following, presented in order from the ground surface downward: urban fill, organic deposits, marine (clay) deposits, and glacial deposits overlying bedrock (Cambridge Argillite) of varying quality. The depth to top of bedrock varies from about 16 to 22 m (53 to 73 ft) below ground surface, generally sloping downward towards the northeast.

Bedrock at the site was characterized into units according to classifications previously established for the historic Central Artery/Tunnel Project (CA/T, commonly referred to as “The Big Dig”), which were based on extensive research and testing. The Massachusetts Turnpike Authority Design Policy Memorandum No. 70 Rev. 1 (DPM 70) (GEI 2001) provided design values and descriptions of the Argillite bedrock for categories ranging from B1 to B3 (B4 and B5 for other rock types) based on rock quality, consistency, and degree of weathering. B1 rock consists of completely weathered to soil or severely weathered Argillite; B2 rock consists of extremely to moderately fractured, severely weathered or severely to moderately weathered rock; and B3 consists of moderately fractured to sound, moderately weathered to unweathered Argillite. The CA/T rock classifications and foundation design parameters were used as a starting point to interpret the bedrock conditions at the project site and in development of foundation design recommendations.

Design challenges

The design team faced several unique challenges that required a collaborative effort to advance the tower design without significantly impacting the project schedule and budget. One of the many challenges was understanding and designing around the existing site constraints. The southwest corner of the parking garage (the tower footprint) is encompassed by a network of near surface and moderately deep (greater than 3 m [10 ft] below grade) active utilities below City streets (Figure 4). Construction against the existing to-remain garage foundations, located along the northern and eastern limits of the future tower, also required special consideration to protect and mitigate undermining and other impacts to the existing concrete-filled pipe pile foundation during new foundation construction. “Lot line” construction near these streets and a selectively demolished garage deterred a conventional “one size fits all” foundation selection approach.

Furthermore, the below-grade space construction also required a support of excavation system to protect the nearby utilities and other existing infrastructure, which had to be married into the aforementioned foundation selection.

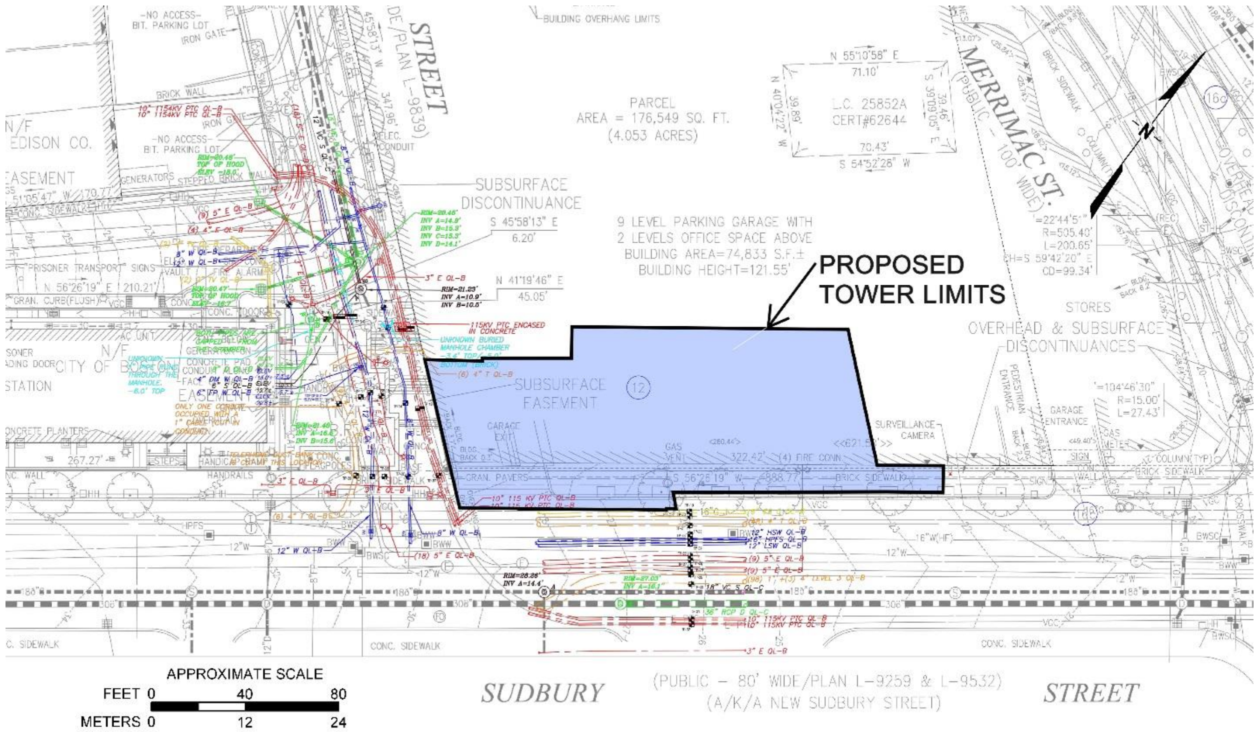

Figure 4. General site plan showing density of existing utilities adjacent to proposed tower.

Several of the existing foundations to remain abut the footprint of the proposed tower, which impacted the selection, design, and location of the future tower foundations. Along the north tower wall, new foundations "threaded the needle," weaving within the column line of existing pile caps. Near the southeast corner of the tower, select columns and foundations were to remain in place as well. The new foundations required permanent casing beyond the anticipated bearing zone of the existing foundations to prevent loss of ground and confinement to the garage foundations during installation of new foundations. Furthermore, the headroom, equipment clearance, and other geometric constraints significantly influenced foundation type selection, as described further below.

Existing garage foundations, former basements, and other urban debris including granite and brick walls, were encountered during excavations for the enabling and tower work and impacted below-grade and new foundation construction (see Figure 5). Due to the anticipated obstructions, significant preclearing excavation efforts were planned to minimize complications created by the shallow subsurface conditions and ensure foundation elements would be installed efficiently.


Figure 5. Buried brick wall exposed during enabling work; soil nail installation in background.

The tower is a tall and narrow-framed building occupying an approximately 1,400 m^2 (15,000 sq ft) area. Due to the slender geometry, the tower column loads were very sensitive to slight changes in structural modeling, from wind/seismic studies to column location adjustments, as building design evolved.

The Structural Engineer developed a finite element model for the structure that was further complicated by the application of geotechnical parameters for the foundation elements. The variation in subsurface conditions, particularly the thickness of weathered bedrock and the depth to underlying competent rock, affected the reaction and resultant stiffness at each column location. Other aspects that notably affected the foundation element stiffness were element type, diameter/size, material type (grout versus concrete) and strength, reinforcing configuration, and the anticipated range of loading. The complexity of both substructure and superstructure required intricate coordination between the geotechnical and structural design.

Complex collaboration and foundation design

Space Limitations

The design process involved continuous collaboration among Ownership, the Structural Engineer, the Geotechnical Engineer, the Construction Manager, the Specialty Foundation Contractor, and the Architect. Based on preliminary cost and schedule evaluations, the project team agreed the tower should be designed to bear on drilled-shaft (caisson) elements, deriving their capacity in bedrock. Design column loads ranged from 8,900 to 58,000 kN (2,000 to 13,000 kips) in axial compression, with the lateral and uplift structural demands depending on the various wind and seismic conditions being evaluated.

Upon assessment of the selectively-demolished garage as-built surveys and contractor design-assist information on equipment clearances and limitations, it became clear that drilled shafts at the necessary design diameter were infeasible at certain locations due to space limitations between existing pile caps where the tower abutted the existing structure. Because temporary and permanently cased elements were required to protect garage foundations as aforementioned, the most practical foundation alternative available was drilled-in micropiles (DMPs). However, the column loads and space limitations precluded axial capacities typical to the Greater Boston area in similar geology (approximately1800 to 2,200 kN [400 to 500 kips]). An extensive load testing program was performed to justify the use of higher-than-normal capacity DMPs for this project. Limited space between the new foundation locations and the existing garage is shown in Figure 6.

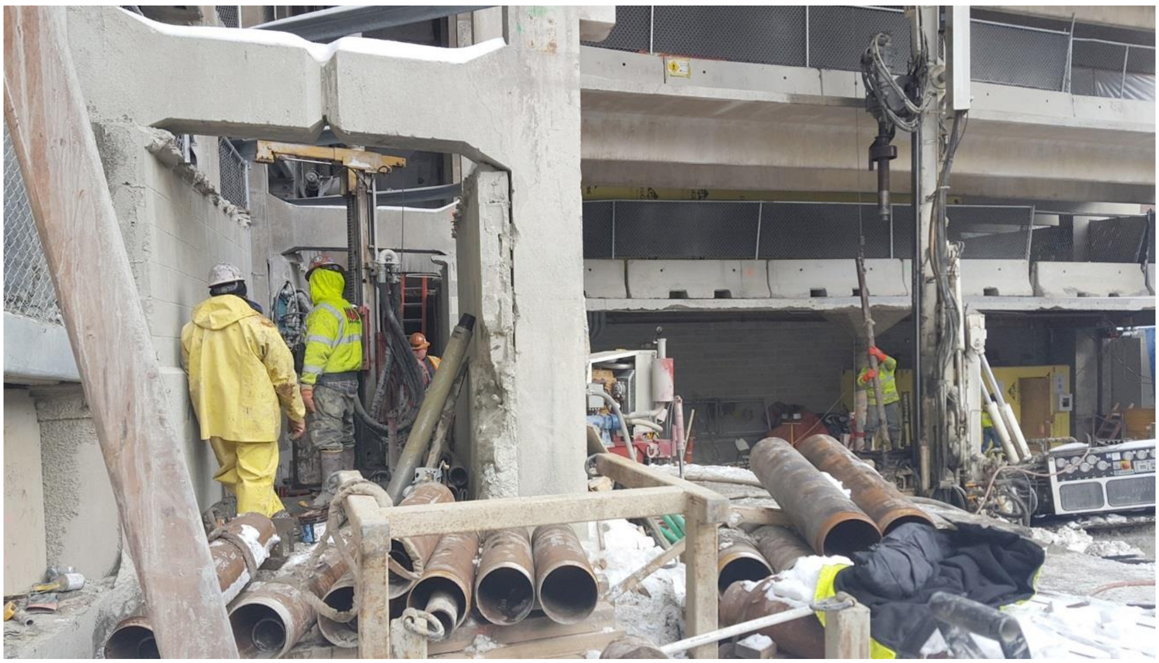

Figure 6. Micropile installation adjacent to the existing garage.

Load Test Program

Four drilled-in micropiles designated TP-1 through -4 with outside diameters ranging from 19 to 27 cm (7-5/8 to 10-3/4 in.) were installed as test piles to further evaluate site-specific rock conditions and to justify higher skin friction resistance values in rock. The micropiles were installed with permanent steel casing through the overburden soils, and the uncased rock sockets varied in elevation and length, targeting specific bond zones in the different bedrock categories. PVC sheathing was used to create a bond breaker in each of the piles to minimize potential load shed and allow for more direct measurements in the rock socket within the targeted zones. Instrumentation included multiple dial gauges, telltale rods, and strain gauges, as shown in Figure 7. The micropiles were subjected to tensile forces applied by jacking against a steel reaction frame. Various rock categories (B1, B2, and B3) were tested to confirm variations in load capacity, and the generalized load displacement curves are shown in Figure 8 (creep holds were removed for clarity).

Figure 7. Instrumented test pile.

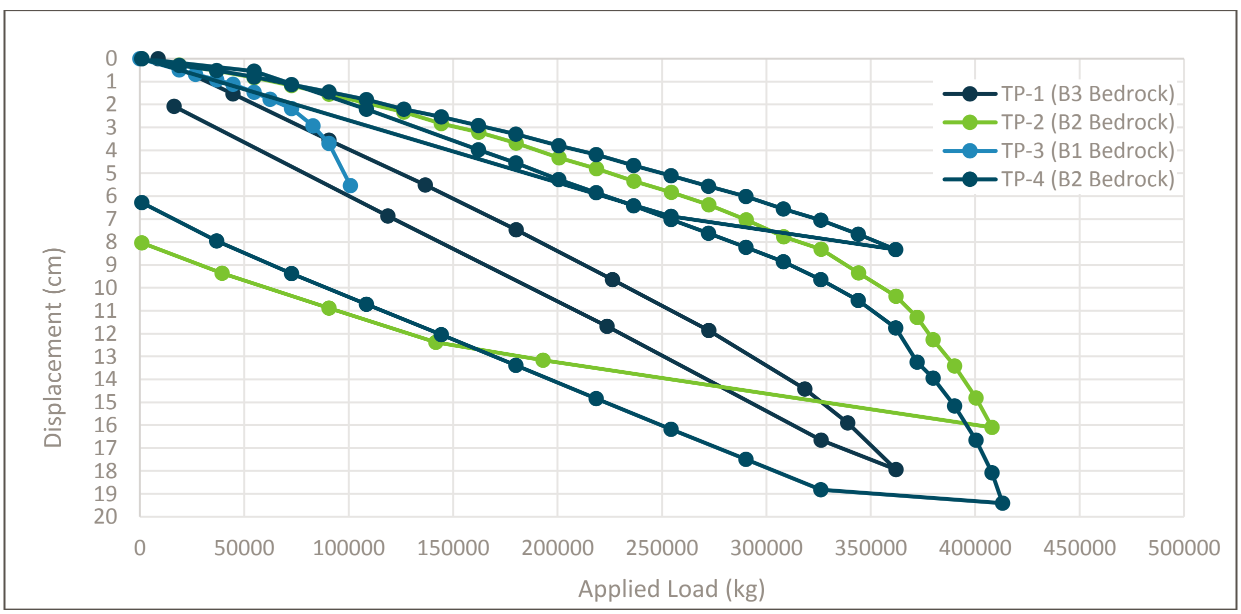

Figure 8. Load test load displacement curves.

Test piles TP-2 and TP-4 were designed to test the skin friction resistance in the B2 bedrock. The test pile results shown in Figure 8 generally show similar load displacement behavior in the B2 bedrock, however test pile TP-4 was subject to cyclic loading conditions.

Based on the load test results, site-specific geotechnical design parameters were developed and are summarized in Table 1. The skin friction recommended for B1 bedrock was not altered from the DPM-70 guidance. Design skin friction in the B2 and B3 rock were reduced for the larger foundation elements (Load Bearing Elements and Drilled Shafts). The skin friction adjustments considered the variability observed in rock quality, the foundation element size and type, the materials (concrete or grout), and the anticipated bond behavior in the bedrock. The micropile grouting process increases the skin friction by filling the fractures and seams in the bedrock within the bond zone.

Table 1. Recommended design parameters for support in bedrock.

Rock Type	Skin Friction Parameters	
	Load Bearing Elements and Drilled Shafts kPa (ksf)	Drilled-in Micropiles kPa (ksf)
B1	167.5 (3.5)	167.5 (3.5)
B2	383 (8)	670 (14)
B3	766 (16)	814 (17)

The load testing program results and design parameters were used to develop foundation recommendations and determine allowable loads for the tower foundation elements. In particular, high-capacity micropiles were designed for maximum design loads of up to 2,900 kN (650 kips), which allowed for foundation construction at the northern edge abutting the existing pipe piles where space limitations and high load demands were present.

Load Bearing Elements vs. Drilled Shafts

After determining drilled shafts were not feasible in all areas of the tower, the design team began considering other foundation alternatives in areas where DMPs were not as cost-efficient considering load magnitudes. Load Bearing Elements (LBEs or Barrettes) were considered to support the tower core and other columns away from the existing foundations. LBEs presented design and cost benefits over drilled shaft foundations, particularly for the tower core. Using continuous LBE foundations to support the tower core would also allow the project team to use the permanent LBE foundations/slurry wall as both the permanent core wall and the temporary support of excavation during core construction. Drilled shaft foundations would require a separate temporary excavation support system and permanent cast-in-place walls. In addition, LBE foundations allowed for a reduction in the overall core cap thickness based on foundation element geometry and its relationship with core wall loads. Based on these considerations and their impact to cost and schedule, the project team selected a combination of high-capacity DMP and LBE foundations to support the tower.

Though LBEs and DMPs were thought to be the final design, early on during construction the debris-laden urban fill presented several difficulties for the Contractor's preclearing activities. "Neat" trench box excavations were not feasible in many areas due to the depth and orientation of intact, remnant granite blocks and other oversize rubble from former structures. Instead, open-cut excavations were required to remove the debris encountered. To minimize impacts to adjacencies during foundation construction near utilities (along the west side of the site), drilled shaft foundations were reintroduced ultimately as a cost-neutral alternative to LBE foundations. HUB Foundation's versatility allowed for drilled shaft installation utilizing much of the same equipment already mobilized on site. With temporary casing until concrete was placed, drilled shafts would better maintain the soils adjacent to the utilities, while also limiting the plan area excavation (1.2 m [4 ft] diameter shaft versus a 2.8 m by 0.9 m [9.25 ft by 3 ft] rectangular excavation). LBEs and DMPs were still planned to support columns loads elsewhere within the footprint.

Having three different foundation types added complexity to the overall design. Figure 9 shows the locations of the various tower foundations.

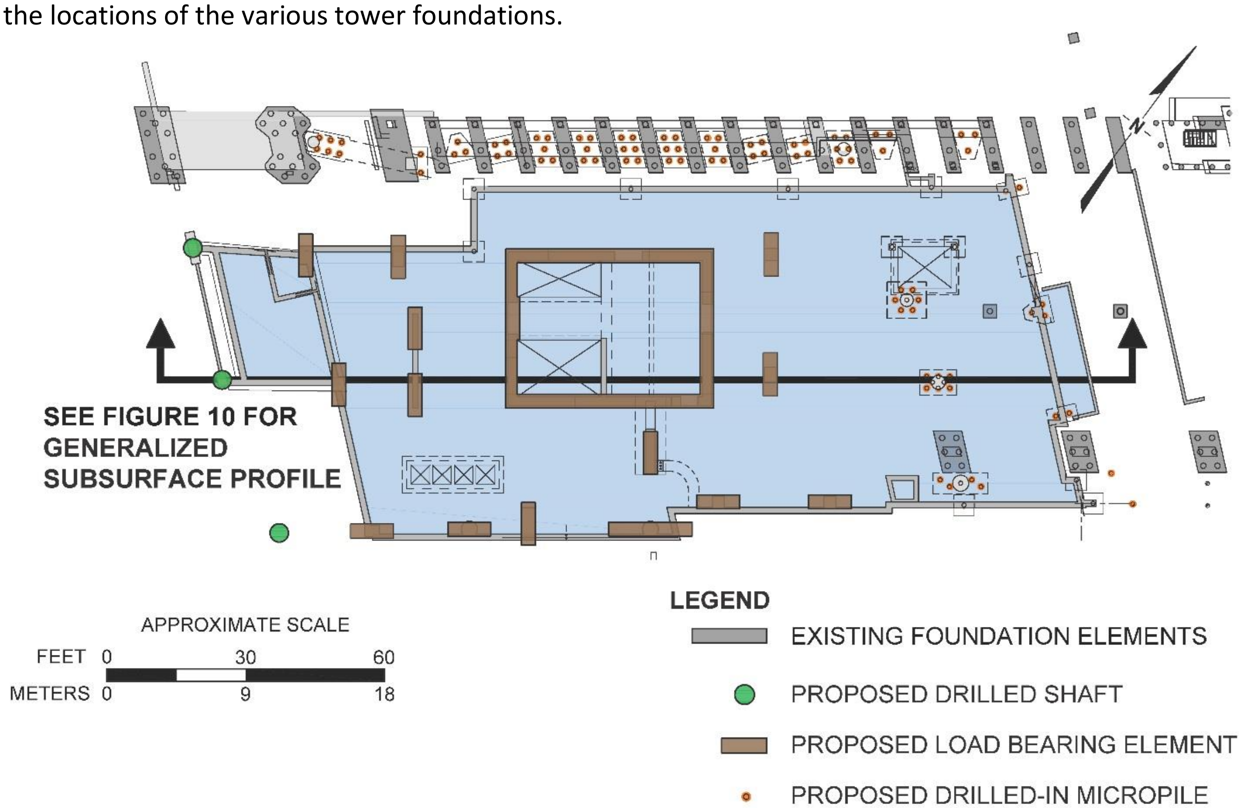

Figure 9. Proposed foundation layout.

Rock Sockets

The variable thickness of weathered rock and depths to reach bedrock affected the individual design embedment lengths for each specific foundation element. Adjustments in foundation locations during the design process required re-evaluation of the recommended rock socket embedment depths. Changes to the foundation type also required re-evaluation of the overall element depth and rock socket length. At some locations, the start of rock sockets was also influenced by the proximity to existing foundations. New DMPs adjacent to existing foundations were designed to protect the existing foundations by permanently casing through the glacial till and completely weathered bedrock and socketing into the underlying sound bedrock. Figure 10 shows a generalized subsurface profile (and foundation elements) oriented east-west through the tower footprint.

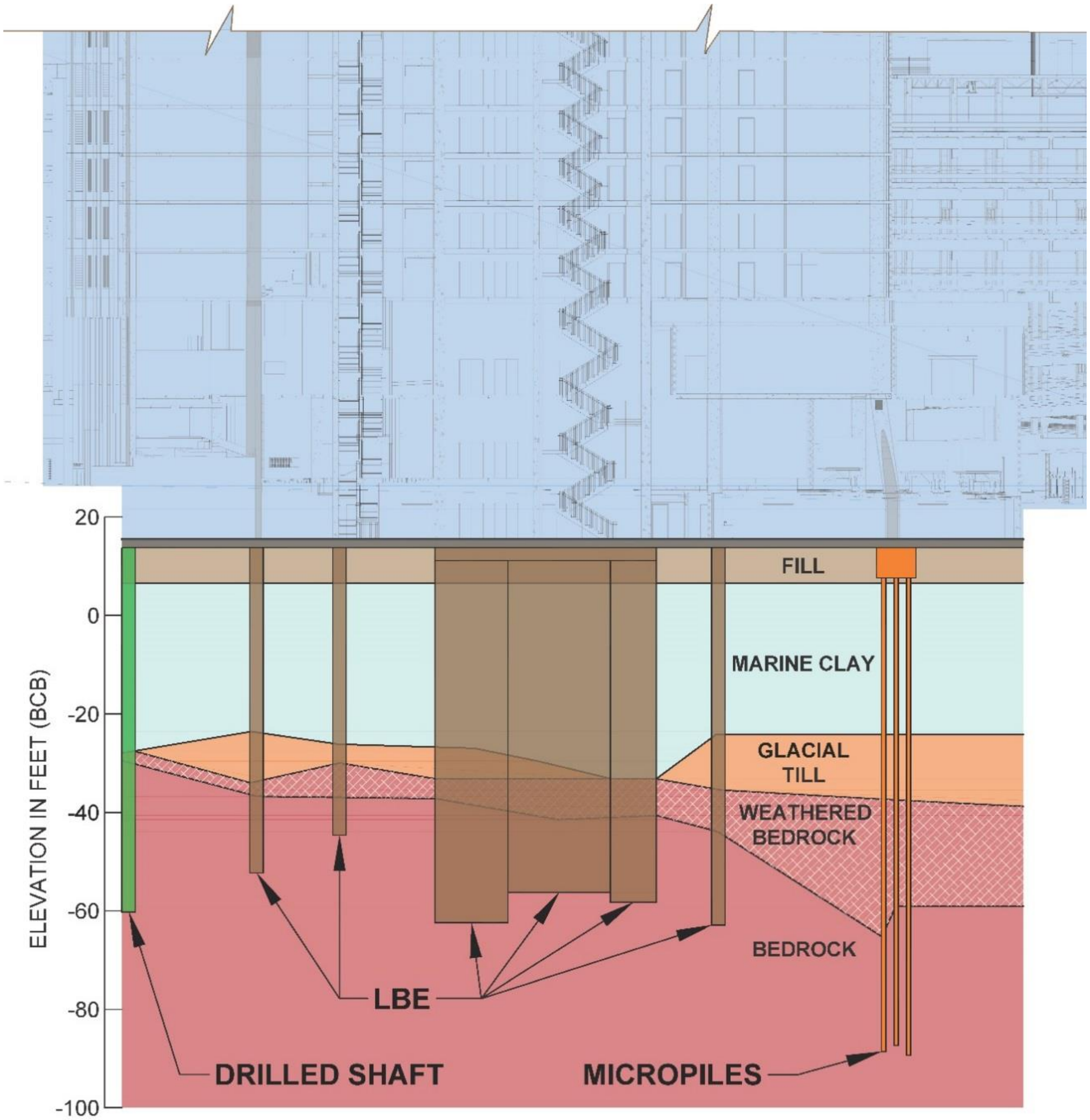

Figure 10. Generalized subsurface profile with foundation elements.

Foundation design embedments were adjusted real-time during installation as the observed stratigraphy varied in depth and thickness from the generalized design profile. Figure 11 depicts multiple concurrent foundation installation operations.

Figure 11. Multiple concurrent foundation operations.

Design Loads

As the design progressed, the Structural Engineer continued to model the structure reactions due to building configuration/loading and foundation selection and resultant stiffness assumptions. The loads were redistributed through the frame in each scenario, and as the column loads were updated, the foundation embedment depths and overall element lengths were adjusted to accommodate the compressive, tension, and lateral load demands. The load test results and other available subsurface data were used to evaluate the axial and lateral capacities of the foundation elements. The adjusted rock skin friction values considerably decreased the lengths of foundation elements, and in doing so affected calculated stiffness values. Several revisions to the foundation design led to an endpoint only after a cooperative evaluation of the structure as described further in the section below.

Soil-Structure Interaction

Structural load distribution through the foundation system was a key challenge in the tower model. The load distribution was heavily influenced by the stiffnesses of each of the various foundation elements, where the larger foundations tended to be stiffer and attract higher loads. The stiffness of an individual foundation element is a function of the foundation element size, materials, length, and stiffness and resistance of the varying stratigraphy. The high-capacity DMPs, LBEs, and drilled shafts each had specific and varying stiffness values based on individual element configurations.

Partial reliance on the existing garage concrete-filled pipe piles made the model more complicated. The structure being somewhat embedded into the existing garage required an understanding of stiffness of the garage pipe piles, which was based on limited available historic records.

In the structural finite element model, foundation stiffness can be modeled as either a foundation or substructure spring representing the combined response of the foundation element and soil at the top of the element/bottom of cap, or as explicit soil springs (horizontal and vertical) along the length of the foundation element (see Figure 12). Stiffnesses were evaluated using finite-element soil-structure interaction analysis software. Load-deflection response values (i.e., spring) for each foundation element were provided to the Structural Engineer as represented in Table 2. This method of evaluation is generally less intensive than discrete soil springs but requires more iteration when the foundation elements are adjusted.

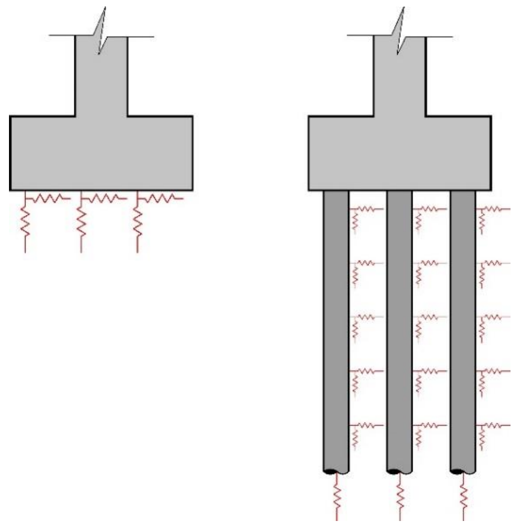

Figure 12. Foundation springs (left) vs. soil springs. (right)

Table 2. Summary of estimated foundation vertical stiffnesses.

Foundation Element	Diameter or Plan Dimensions m (in.)	Estimated Range of Vertical Stiffness kN/m (kips/in.)
Micropiles	0.27 (10.75)	1.4×10^5 - 1.9×10^5 (800 - 1,100)
LBEs	0.9 x 2.8 (36 x 110.4)	2.5×10^6 - 2.8×10^6 (14,000 - 16,000)
Tower Core LBEs	0.9 x 2.8 - 0.9 x 6.6 (36 x 110.4 - 36 x 261.75)	3.5×10^6 - 7.0×10^6 (20,000 - 40,000)
Tower Core Corner LBEs	0.9 x 7.2 - 0.9 x 7.8 (36 x 282 - 36 x 308.5)	6.3×10^6 - 7.4×10^6 (36,000 - 42,000)
Drilled Shafts	1.2 (48)	1.5×10^6 (8600)
Existing Concrete-Filled Steel Pipe Piles	0.4 (14)	1.2×10^5 (700)

The Structural Engineer developed the initial structural model using preliminary geotechnical parameters. Column loads were developed based on these parameters and provided to the Geotechnical Engineer. The rock socket lengths for each foundation type were determined based on these loads and foundation springs were provided (the foundation springs are a function of the element length). An iterative modeling process began and continued throughout the design phase as loads, foundation types, and rock embedment lengths were adjusted. To streamline this process, soil springs were provided to allow the Structural Engineer to more independently model the foundation reactions. The structural finite element model incorporated the foundation springs for isolated foundation elements. In the core and shear walls the model used a combination of foundation springs above the rock socket and discrete soil springs within the rock socket to account for lateral load requirements. The process of developing soil springs was more labor-intensive and added complexity to the structural model but ultimately reduced the number of iterations. The range in estimated vertical stiffness values for the various foundation elements is summarized in Table 2, and a three-dimensional image of the structural model, provided by the Structural Engineer, is shown in Figure 13.

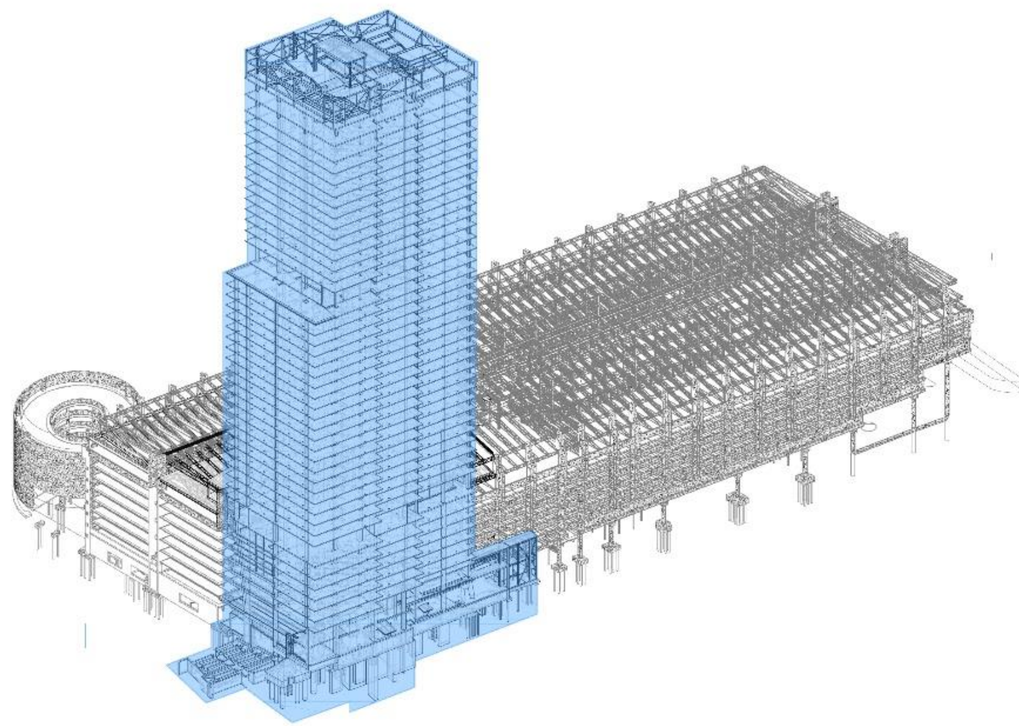

Figure 13. Three-dimensional model of proposed tower and existing garage.

Conclusions

The success of this challenging project was made possible by the interaction among the project design team members, where an accelerated schedule required communication early on and continuously as the design evolved, the constructability of certain foundation types was vetted, and cost optimization was studied. Other key elements contributing to the overall success of the project are summarized below:

- Drilled-in micropile load testing allowed for higher than typical capacity micropiles to be used for foundation support where other foundation alternatives were not feasible due to space limitations and the proximity to existing foundation elements.
- The load test results were used also to develop site-specific geotechnical design parameters for the various strata, which helped verify increased skin friction values, thereby reducing rock socket embedment depths for DMPs and other employed foundation elements as well.

- A variety of foundation types were incorporated into the foundation system to accommodate the site constraints, column loads, and structural design, and required input from multiple disciplines and trades.
- Variations observed in the subsurface conditions or in existing site conditions during foundation excavation were met with real-time collaborative evaluation (foundation embedment lengths, plan location shifts, anomalous conditions that created significantly shorter or longer elements, etc.).
- Collaboration between the Structural Engineer and the Geotechnical Engineer required expertise in soil-structure interaction to successfully implement the tower design.
- Open communication amongst the entire project team, led and empowered by Ownership, enabled the early realization of the complexity of this project and allowed the design to evolve and construction to be completed ahead of an accelerated construction schedule.

References

Massachusetts Transportation Authority (2001). *Design Policy Memorandum No. 70 Rev. 1, Guidelines for Structural and Geotechnical Analysis of Cut and Cover Tunnels and Drilled Shafts.* (July 11, 2001).