Everything but the Kitchen Sink:

Use of Multiple Foundation Types to Allow for Construction on a Boston Hillside

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Abstract

This paper presents a case history of an urban re-development project in Boston, Massachusetts, where multiple foundation types were used to overcome the unique geotechnical and environmental subsurface conditions present at the site. The project site is located on a hilltop adjacent to a hospital and a high school. The majority of the site has a thin layer (1 m [3 ft]) of fill overlying shallow glacial till deposits. However, there are areas of the site where the fill is up to 8.8 m (29 ft) thick and contains debris including Asbestos Containing Materials (ACM). To allow for the construction of two of the planned buildings over the deep fill areas, a hybrid foundation solution was proposed, consisting of drilled micro-piles and shallow foundations constructed both on natural soils and on the existing fill following the installation of ground improvement. In addition, a variable-height permanent soldier pile and lagging retaining wall was required to achieve the planned site grades.

Introduction

This paper is a case study of a re-development project in Boston, Massachusetts, consisting of the construction of four new residential buildings on the site of a former church and monastery. The site is located in the Brighton neighborhood of Boston in an area close to several major universities and hospitals, making it a desirable location to develop rental housing.

The proposed site layout, along with the location of the former St. Gabriel's church and monastery which is to remain, is shown in Figure 1 below. Due to the variability of the subsurface conditions and the presence of Asbestos Containing Materials (ACM) in portions of the existing fill soils, a hybrid foundation system consisting of a mix of deep and shallow foundations was recommended to facilitate construction of the new buildings. A significant retaining wall was required to level the northern portion of the site to achieve the planned finished site grades.

This paper specifically focuses on the foundation challenges associated with the construction of Buildings 2 and 3 that are located above a historically filled slope.

Elevations referenced in this paper are in meters and refer to Boston City Base datum.

Figure 1. Conceptual Site Plan (courtesy of Cabot, Cabot & Forbes)

Site conditions and history

The site, located along Washington Street in the Brighton neighborhood of Boston (Figure 2), is located on a hilltop (historic glacial drumlin). The site grades vary by almost 15.2 m (50 ft) ranging from El+43.6 (El +143 ft) near Washington Street and up to El+58.2 (El +191 ft) near the existing Church. Steep slopes exist on the north (approximately 2H:1V) and northwest (approximately 1H:1V) portions of the site and grade from approximately El +56.4 (El +185 ft) down to approximately El +45.7 (El +150 ft) on the adjacent properties. A reinforced concrete parking garage associated with the adjacent hospital is located to the west of the site, and a public high school and a heating and cooling plant building associated with the hospital are located along the northern edge of the site. The eastern side of the site is bordered by a mix of residential and commercial buildings. Figures 3 and 4 show the existing slopes to the northwest and north of the project site, respectively.

The site was first developed in 1911 when the existing Monastery building was constructed followed by the existing St. Gabriel's church in the early 1920s. As noted on Figure 2, the parking area to the north of the existing church building was expanded over time as the existing slope was filled in various phases. Figure 5 shows a historic photograph of the site during the early stages of the filling. The approximate limits of the filling sequences are shown on Figure 2. Based on a historic newspaper article (Lorant 1963), the filling that occurred in the 1960s consisted of soil and debris from the adjacent hospital expansion project.

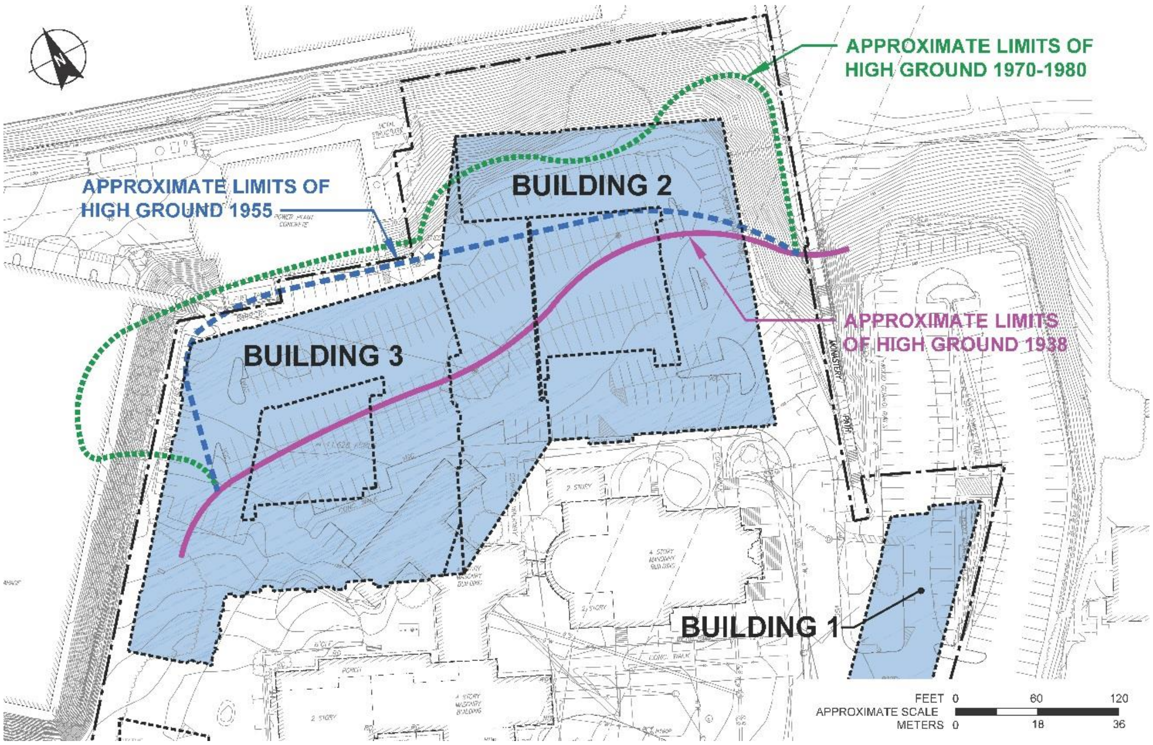

Figure 2. Existing Conditions Plan

Figure 3. Existing slope along northwest side of site (looking south)

Figure 4. Existing slope along the north side of site (looking west)

Figure 5. Historic Photograph (undated, courtesy of Cabot, Cabot & Forbes)

The adjacent parking garage (pictured in Figure 3) was constructed in 1991, along with the engineered slope. The parking garage is believed to be founded on shallow foundations bearing on the natural glacial soils or on compacted granular fill placed following the removal of the then existing fill.

The heating and cooling plant was constructed in 1999. The building was designed to step into the existing slope and is founded upon spread footing foundations placed either on natural soils or on compacted granular fill placed after the removal of the existing fill.

Development

The proposed development consists of the construction of four new residential buildings, ranging in height between five and six stories, that will surround the existing St. Gabriel's Church and Monastery (Figure 1). As previously stated, this paper focuses on the areas of Buildings 2 and 3, and thus information on Buildings 1 and 4 will not be presented.

Buildings 2 and 3 have a combined footprint of approximately 8,825 sq m (95,000 sq ft) and are connected by a below-grade parking level underneath both buildings. The structures consist of a steel podium level with up to five levels of wood frame structure above. The lowest level slab (parking surface) is located at approximately El +55.0 (El +180.5 ft) (which required a cut of approximately 0.6 to 1.5 m (2 to 5 ft) across the building footprint).

Additionally, the existing slope along the north side of the site will be filled to further expand the buildable area for the new building.

Subsurface information

To explore the subsurface conditions at the site, a program of test borings and test pits was undertaken at the site. Data from the exploration programs indicate subsurface soil conditions generally consist of miscellaneous fill over dense glacial till soils above bedrock. The test borings were generally advanced through the fill soils and into the glacial soils. Several of the borings were terminated on bedrock, and the bedrock was cored in two locations. The test pits were advanced within the footprint of Buildings 2 and 3 to explore the composition of the fill soils.

The miscellaneous fill was highly variable but generally consisted of brown to gray, medium dense to dense silty SAND or sandy SILT with gravel and varying amounts of clay, cobbles, cinders, coal, ash, and organic materials, with significant debris layers. The thickness of the fill layer ranged from approximately 0 to 8.8 m (0 to 29 ft). The thickest fills (3.7 to 8.8 m (12 to 29 ft) were encountered on the north and west sides of the existing church in the paved parking areas (Buildings 2 and 3). The fill contains oversized rocks, construction debris, and varying amounts of non-soil materials. The area with most significant amount of debris and oversize materials was located to the north of the blue line shown on Figure 2 above.

Figure 6 below shows pictures of the fill and debris encountered within the test pits conducted within the footprint of Buildings 2 and 3 to the north of the 1955 fill line. Buried construction debris within the test pits was noted to consist of large pieces of concrete, some exceeding 1.2 m (4 ft) in dimension.

Figure 7 shows pictures of the fill and debris encountered within the test pits conducted within the footprint of Buildings 2 and 3 in areas that were filled between 1938 and 1955.

Glacial till was encountered below the fill and was generally described as a dense to very dense light brown sandy SILT with gravel. Frequent cobbles were noted in the boring logs and boulders are also present in this unit. Generally, the depth to glacial till decreases from north to south across the site, ranging from 9.4 to 0.6 m (31 to 2 ft). Where penetrated, the glacial till was found to be approximately 7.6 m (25 ft) thick.

Bedrock, consisting of moderately hard to hard, freshly to slightly weathered conglomerate, was encountered below the glacial till.

Groundwater was encountered at approximately El +44.8 (El +147).

Figure 6. Fill encountered within Building 2 and 3 north of the 1955 fill line.

Figure 7. Fill encountered within footprints of Buildings 2 and 3 between 1938 and 1955 fill line.

Environmental constraints

During the test pit investigations, suspect ACM materials were encountered within the fill and later confirmed by laboratory testing. Local environmental regulations require that any soil disturbing activities within soils containing ACM debris must be completed by a licensed abatement contractor under the provisions of a site-specific work plan filed with the Massachusetts Department of Environmental Protection (MassDEP) that outlines specific worker health and safety protocols and air monitoring requirements.

The site-specific work plan also imposes additional work restrictions such as limiting the size of stockpiles, strict dust control during construction (which prohibits certain types of construction such as mechanical screening of fill to remove debris or micro-pile installation using compressed air) and requires clean cover to be constructed to limit future exposure to the ACM debris in soil. Additionally, off-site disposal of soil containing ACM debris is four times more expensive than the disposal of typical urban fill.

Within the Building 2 and 3 footprint, the top 1.5m (5 ft) of soil below the existing grades was found to be non-ACM impacted. The northern two thirds of the Building 2 and 3 footprint contain ACM-impacted soils and the southern third was considered non-ACM impacted soils (as defined in the MassDEP work plan). The presence of the ACM soils, the associated required environmental protocols (which resulted in premium costs), and sensitive abutters (hospital and high school) were driving factors in determining the best foundation approach.

The Hybid Foundation approach

The conditions across the Buildings 2 and 3 site vary significantly from south to north with fill thickness ranging from 2.1 to 8.8 m (7 to 29 ft). In addition, the fill below the middle and northern portion of Buildings 2 and 3 was noted to contain various amounts and sizes of debris and ACM, starting generally at a depth of 1.5 m (5 ft) or greater below ground surface. Figure 8 shows a schematic cross-section (south to north) of the subsurface conditions through Building 2.

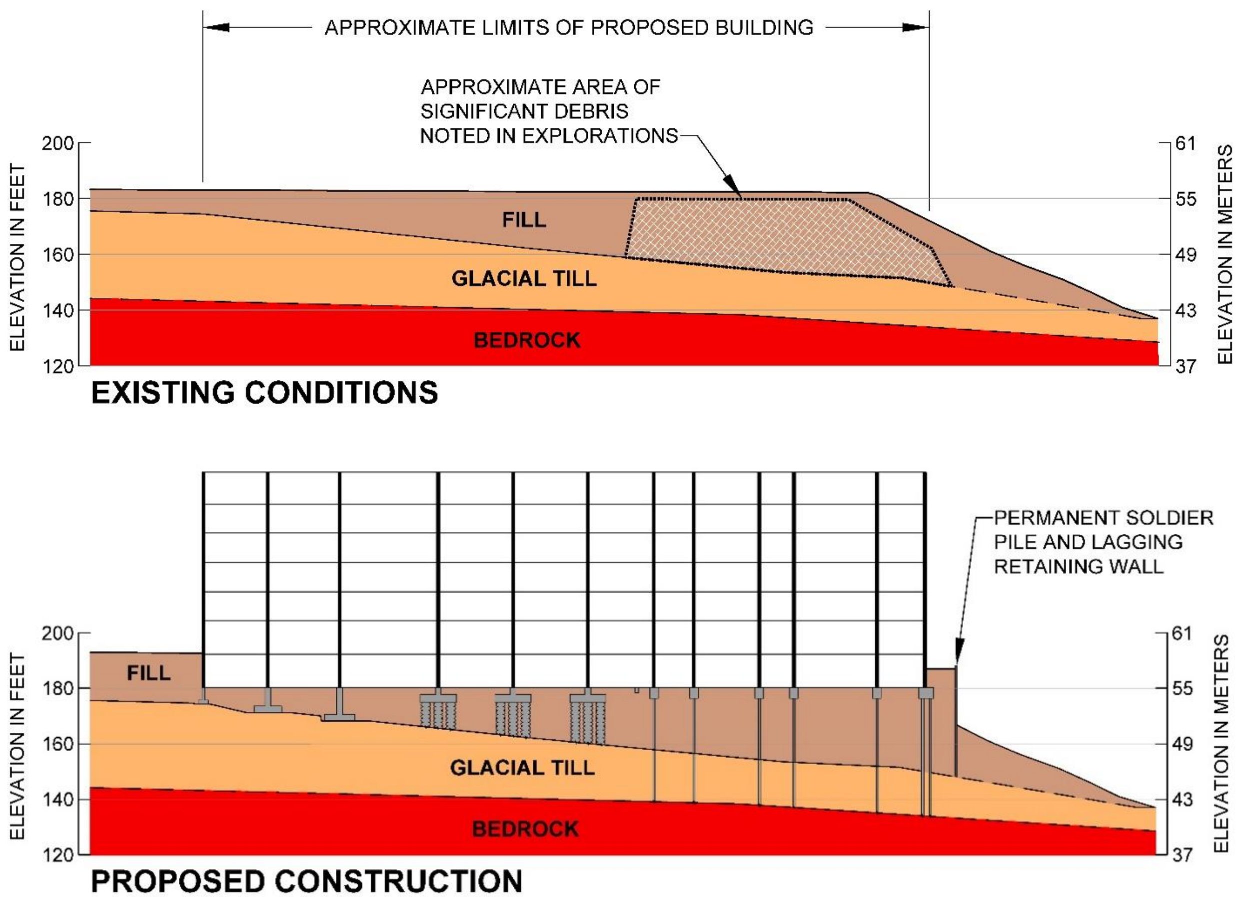

Figure 8. Cross-section: a) existing conditions b) proposed foundation conditions.

Several deep and shallow foundation options were evaluated by the project team, including driven piles, pressure injected footings (PIF), micro-piles, ground improvement to improve the existing fill, and traditional excavation and replacement of the existing fill.

Driven piles and PIFs were not considered feasible due to the amount and size of the debris in the deeper fill areas. For pile/PIF installation, pre-clearing to remove obstructions to depths of more than 8.8 m (29 ft) would have been required. The size of the excavations would have been very large to conduct them as open cut excavations, and the environmental restrictions on stockpiling and screening of the soils would have made it difficult to handle, cull debris, and replace the existing soils back in the excavation. Additionally, there are vibration-sensitive structures (high school and hospital facility) adjacent to the site.

Traditional excavation and replacement of the existing fill within the ACM-impacted soils area was not considered feasible due to the expense and logistics of handling and disposing of the ACM soil in accordance with the MassDEP workplan.

Micro-piles are a technically feasible alternative as they can be advanced through the existing fill, including the debris, to the underlying bedrock with minimal spoils and do not require extensive pre-clearing of locations. However, the cost of installing micro-piles for the entire building was cost prohibitive.

Installation of ground improvement [Rammed Aggregate Piers (RAPs), Grouted Aggregate Piers (GAPs), etc.] to improve the existing fill soils to allow for the use of shallow foundations was considered feasible in the fill areas where significant debris was not observed (south of the 1955 fill line – Figure 2). However, use of ground improvement in the deeper fill area (north of the 1955 fill line) was not considered feasible due to the presence of the larger debris that would require pre-clearing at the pier locations.

Therefore, a hybrid foundation solution was required to control project costs, comply with the local environmental restrictions and sensitive abutters, and limit impacts to the project schedule. Based on input from the design and development team and the Construction Manager, the most economical foundation system meeting design and performance criteria was a hybrid system consisting of:

1.) conventional spread footings (constructed outside of the ACM-impacted soil zone),
2.) footings constructed after improving (stiffening) the existing fill soils, and
3.) micro-pile foundations.

Figure 9 shows a diagram with the approximate limits of the three foundation systems within the limits of Buildings 2 and 3.

The approximate limits of each of the three foundation approaches were selected based on the subsurface conditions encountered. Micro-piles were selected for the deepest fill area where significant and sizable debris was encountered, because they could be advanced through the debris with minimal pre-excavation and disturbance to the ACM-impacted soils.

Stiffening the soils with ground improvement in the middle fill areas was selected to minimize excavation within the ACM-impacted soils and also to reduce costs, as installation of micro-piles in this area would have added significant cost to the project.

Traditional excavation and replacement was selected for the southernmost area as it was located outside the ACM zone. Thus, the MassDEP plan earthwork restrictions do not apply, and the depth to suitable natural bearing soils (glacial till) was less than 1.5 (5 ft) below the normal bottom of footing elevation. As a result, traditional excavation and replacement of the existing fill was less costly than ground improvement for this area.

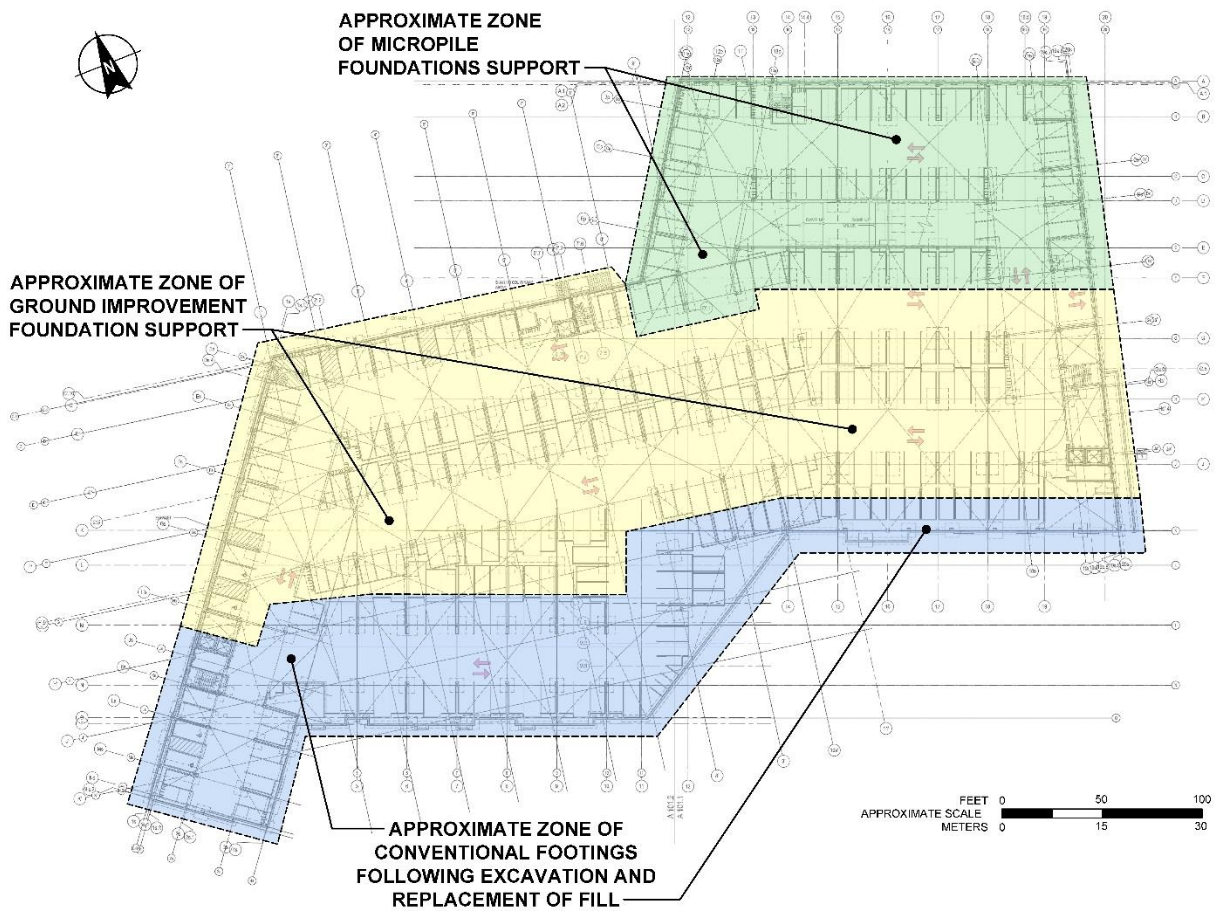

Figure 9. Approximate Building 2 and 3 Foundation Zones

From a settlement perspective, the three foundation alternatives were designed to perform similarly. The allowable bearing capacity of the shallow foundations on natural glacial till soils was established to limit settlements to approximately 2.5 cm (1 inch) of total settlement. Specifications for the ground improvement incorporated design criteria to limit settlements of the improved zone to 1.9 cm (0.75 inch) and the micro-piles are anticipated to elastically compress on the order of 2.5 cm (1 inch) or less.

Due to the presence of voids encountered in the fill within the micro-pile foundation area, the lowest level slab is a structural slab designed to span between pile caps. The lowest level slab in the ground improvement area and the conventional footing area is a conventional slab-on-grade. The fill in these areas was noted to be generally granular in nature and medium dense or better. To improve the slab performance, the top 0.6 m (2 ft) of soil below the planned slab subgrade will be removed, the resulting subgrade proof-compacted, and the excavated materials re-placed and compacted in controlled lifts. A tipping slab is planned between the structural slab and the slab-on-grade areas to control any minor differential slab settlements.

Retaining wall

As shown on the cross-sections in Figure 8, the existing slope to the north of Building 2 was filled to increase the buildable area of the site. The filling was required to achieve a level grade around the structure. The local building code uses the average grade on all four sides of the structure to determine the allowable building height, and thus the grade along the north side of the building had to be raised to allow for the structure to achieve the planned maximum height.

Slope stability evaluations of the existing slope revealed that for static conditions the existing slope had a factor of safety of approximately 1, and a factor of safety of less than 1 for seismic conditions.

To raise the grade on the north side, a retaining wall with an exposed face of up to 8.5 m (28 ft) was required. A traditional mechanically stabilized earth (MSE) wall would have been the most economical solution; however, given the slope stability issues and the presence of up to 6.1 m (20 ft) of uncontrolled ACM-impacted fill below the wall alignment, it was determined that an MSE wall would not have been stable on the slope without the removal and replacement of the existing fill. To allow for the required raise in grade and to also increase the overall factor of safety of the existing slope, a permanent soldier pile and lagging wall (with piles extended through the fill and into the underlying glacial till) was designed. Part of the wall was designed as a cantilever, and part of the wall was designed to be laterally braced with deadman anchors positioned below the new building to provide additional support.

The installation of the wall also allows for the re-use of additional ACM-impacted soils (generated from pile cap, footing, and utility excavations) behind the wall to raise the grade, thus saving off-site transport and disposal costs.

Summary

The need for additional housing to serve the City of Boston community is great. Due to a strong economy, active development market, urban locations, site constraints, and previous development impacts, sites available for development in the Boston area without significant site work challenges are often hard to come by. Creative solutions are required to maximize the developable area of the site and address foundation performance and cost challenges. In the case study discussed above, both geotechnical and environmental challenges were solved using a hybrid foundation approach that resulted in a cost-effective re-development of a prime site location in Boston. A combination of micro-piles, ground improvement, and shallow foundations were engineered to support this ambitious redevelopment effort.

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- Structural Engineer – SCA Engineers
- Geotechnical Engineer – Haley & Aldrich
- Excavation Contractor – J. Derenzo Company

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