Micropiles Counter Unbalanced Loading During Partial Building Demolition

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ABSTRACT

A new facility was constructed to replace the existing VCU Children's Hospital of Richmond Pavilion. The existing building consisted of two full floors below grade with one floor above grade on the south side and four floors above grade on the north side. As part of the proposed construction of the new Pavilion, the southern portion of the existing Pavilion was demolished and replaced with a new structure with four below-grade levels. This demolition and excavation resulted in unbalanced lateral earth pressures on the remaining portions of the building, and affected the structure's wind and seismic resistance. In addition, the portion of remaining structure adjacent to the new excavation would need to be underpinned to prevent instability to the 2 additional below grade floors. To remedy this condition, a system of concrete shear walls and buttresses were constructed inside the existing building to provide the required building bracing for lateral loads. A network of tiebacks and micropiles was conceived to provide the necessary lateral and vertical support during demolition and excavation. This paper discusses the rationale for selection of this system and describes the general design procedure followed, which involved tight collaboration between the structural and geotechnical engineer. Construction and quality control are also discussed.

INTRODUCTION

The 59,500 sq m (640,000 sq ft) VCU Health Children's Hospital of Richmond Pavilion is the largest, most advanced outpatient facility dedicated to children in the region and serves as a gateway to the VCU Medical Campus in Downtown Richmond, Virginia (Figure 1). The Children's Pavilion was redesigned from its existing four stories above grade and two below (Figure 2), to 11 stories above grade and 4 stories below grade, and encompasses more than two-thirds of a city block. The new construction extended 16.8 m (55 ft) below Broad Street, or 10.7 m (35 ft) deeper than the lowest grade of the existing Pavilion. The design needed to include potential future expansion of up to seven more floors above grade and expansion of the below-grade levels to the north. Additionally, the existing pavilion had to be kept operational throughout construction. Dunbar Milby Williams Pittman & Vaughan (DMWPV) provided structural engineering and Schnabel Engineering provided geotechnical and geostructural engineering for this significant project. The two firms collaborated on the design of the structural support and underpinning of the existing adjacent pavilion building. Nicholson Construction was the specialty subcontractor for the installation of micropiles and tiebacks.

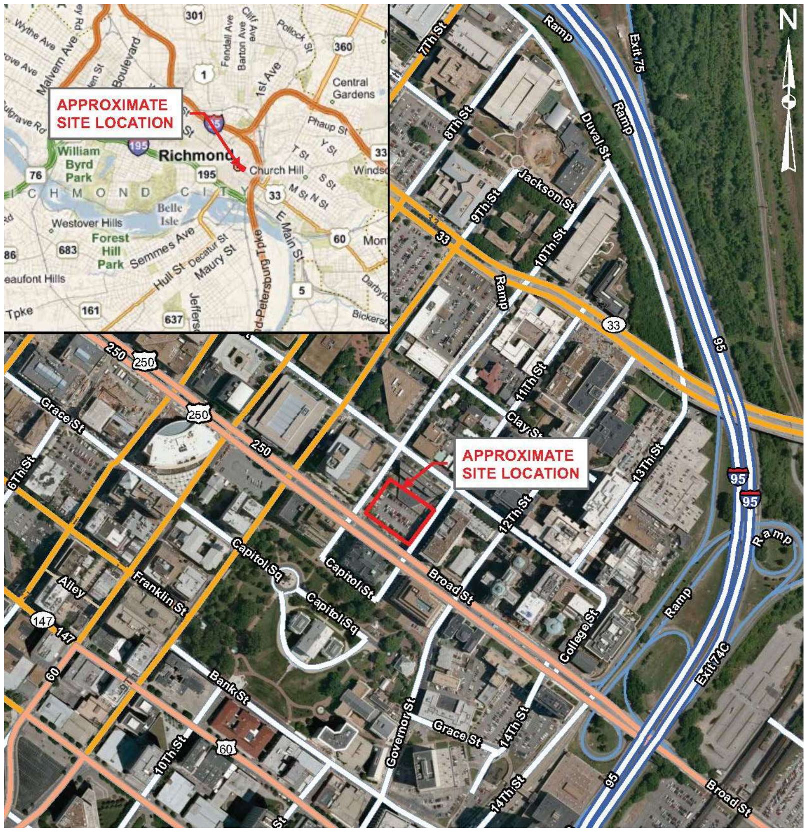

Figure 1. Site vicinity

Figure 2. Existing Building Prior to Demolition (credit Elmquist)

The first phase of construction for the Pavilion included demolishing the southern half of the existing Children's Pavilion (cut along Column Line D), see Figure 3, and installing the new Support of Excavation (SOE) pit south the existing building cut plane. The urban environment, deep excavation and necessity of keeping the Pavilion in service throughout construction created many structural, geostructural and geotechnical design challenges, chiefly: working space was limited and cluttered with utilities; deflection of nearby structures had to be limited to a very small magnitude; the foundation design had to account for future vertical expansion; and the time that the excavation was open had to be kept to a minimum.

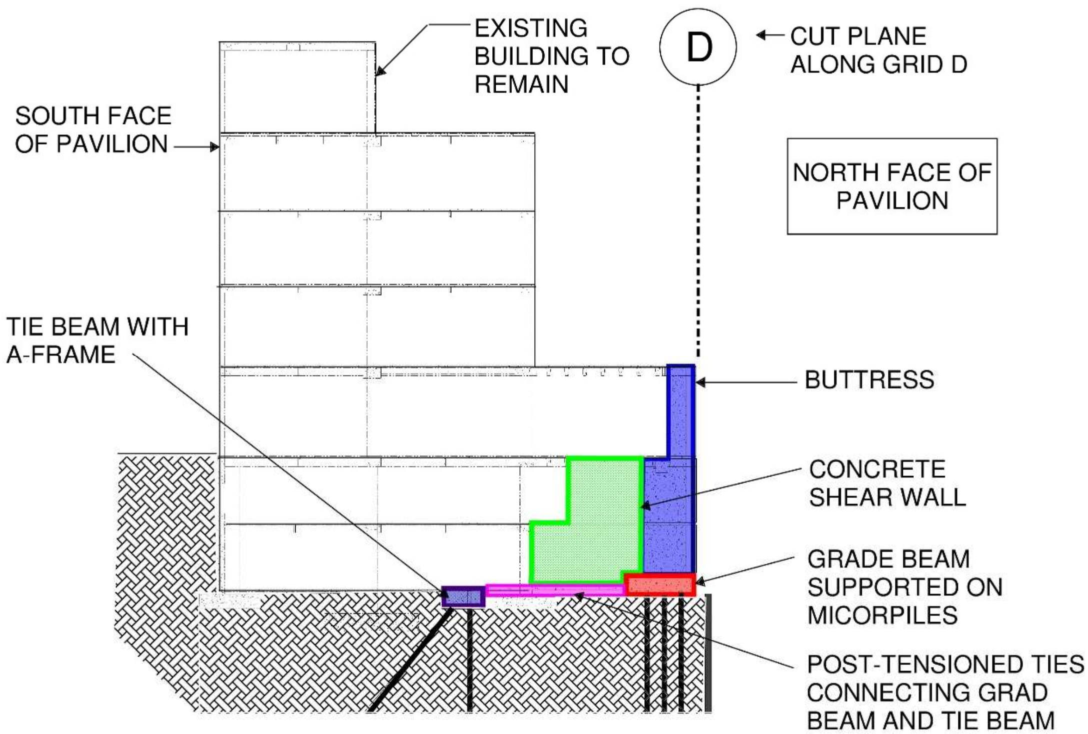

Figure 3. Elevation View of System

GEOLOGY

The geologic stratigraphy in downtown Richmond typically consists of Pleistocene Age terrace deposits, Miocene Age Calvert Formation, Eocene Age sands, Cretaceous Age sand and gravel, and residual soils and rock of the Petersburg granite formation. The terrace deposits are alluvial soils that typically consist of a mixture of clay, silt, sand and gravel exhibiting moderate strength and compressibility. The Calvert Formation consists of marine-deposited sediment. These soils exhibit moderate to high strength and low to moderate compressibility. The Eocene and Cretaceous Age soils are also marine sediments that typically exhibit high strength and low compressibility. Residual soils are derived from the chemical and physical weathering of the underlying parent material, the Petersburg granite rock.

Figure 4 shows the stratigraphy on site. The existing parking lot portion of the site has several feet of existing fill below the ground surface. Samples of the fill contained debris suggesting one or more structures were, at one time, located within this area of the site. This observation is supported by the Environmental Impact Report for this project prepared by others which references multiple buildings previously occupying the site including the Broad Street Methodist Church. Several borings encountered concrete obstructions at the bottom of the existing fill. These obstructions may indicate that the now-demolished structures had basements. Table 1 shows the properties considered for each soil layer.

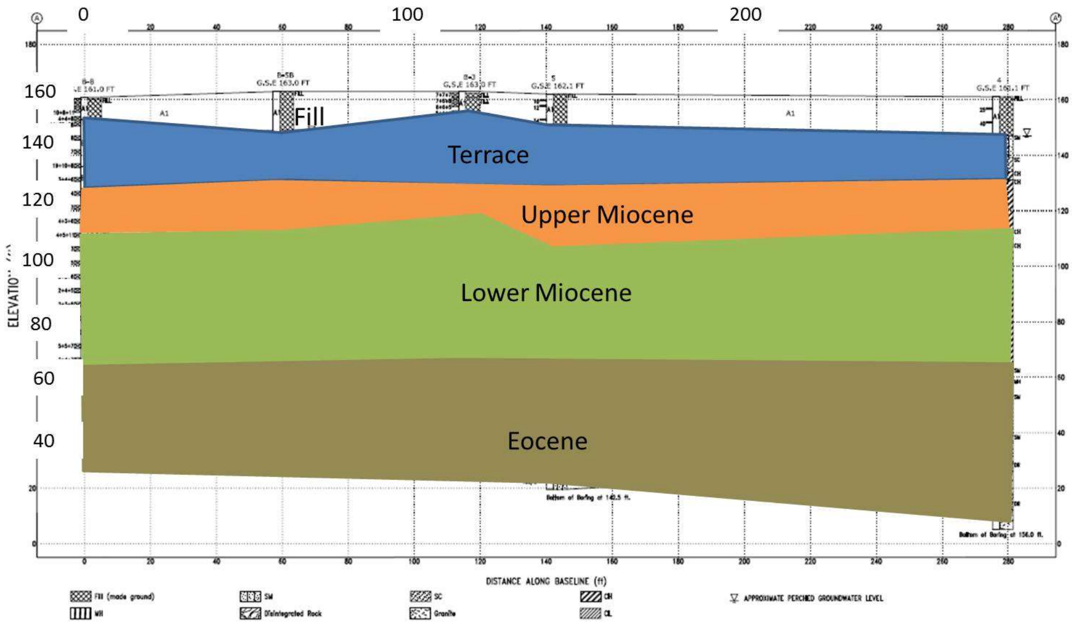

Figure 4. Site Stratigraphy

Table 1. Cohesionless and Cohesive Soil Properties

Name	γ_{unsat} kN/m^3[pcf]	γ_{sat} kN/m^3 [pcf]	ν' [-]	E' MPa [psf]	c' [psf]	ϕ' [°]	K_o [-]
Stratum A1 - Coarse-Grained Fill	18.8 [120]	19.3 [123]	0.35	9.6 [200,000]	0	32	0.5
Stratum B1 - Coarse-Grained Terrace	18.8 [120]	19.3 [123]	0.35	19.2 [400,000]	0	35	0.5
Stratum E - Eocene	20.4 [130]	20.9 [133]	0.35	23.9 [500,000]	0	40	0.5
Stratum F- Cretaceous	21.2 [135]	21.7 [138]	0.35	35.9 [750,000]	0	42	0.5
Stratum G - Disintegrated Rock	22 [140]	22.5 [143]	0.35	47.9 [1,000,000]	0	45	0.5

Name	γ_{unsat} kN/m^3[pcf]	γ_{sat} kN/m^3 [pcf]	Cc [-]	Cs [-]	c' kPa [psf]	ϕ' [°]	Su kPa [psf]
Stratum B2 - Fine-Grained Terrace	18.1 [115]	18.5 [118]	0.39	0.015	4.8 [100]	30	96 [2,000]
Stratum C2 - Fine-Grained Upper Miocene	18.1 [115]	18.5 [118]	0.39	0.015	4.8 [100]	30	120 [2,500]
Stratum D - Fine-Grained Lower Miocene	15.7 [100]	16.2 [103]	0.525	0.020	12 [250]	31	239 [5,000]

STABILIZATION OPTIONS AND LOAD PATH

The existing structure that would remain in place was modeled using Autodesk Revit and REM FEM analysis software. Multiple stability options were considered, including new internal concrete shear walls, new steel braced frames, exterior steel trusses on the north face, new exterior cast-in-place augercast piles integral to a new retaining wall on the north face, and combinations of each of these solutions. The best option was not solely cost-driven. The owner's desired use of part of the structure during construction, encroachment limits due to easements on the northern exterior of the structure, and minimizing disruptions to the surrounding structures were all taken into account. It was determined that the best option to support lateral loads on the existing building during the partial demolition and adjacent excavation would be seven new 2.7 m by 2 m (9 ft by 6.75 ft) full height concrete columns along the cut plane, each with an integral 0.6 m (2 ft) thick concrete shear wall perpendicular to the line of demolition. These large columns, referred to as buttresses, would be supported on a continuous grade beam the entire length of the existing structure at the face of the cut plane on Column Line D. Support of the grade beam and buttresses during the adjacent excavation required a pile system designed jointly by the geotechnical and structural engineers (Figure 5).

The structural engineer determined the lateral loads that needed to be resisted at each level and distributed the loading to the seven new concrete buttresses located along the cut plane and their adjacent concrete shear walls. The new buttresses were spaced 7.3 m (24 ft) on center to match the existing grids and columns. Connections to the new concrete elements were designed and detailed. Finally, a continuous grade beam along the cut plane was initially modeled to support the loads. If an adjacent excavation were not required, the grade beam would be sized for the allowable soil bearing capacity. Additionally, soil friction between the grade beam and soil was not sufficient to resist the lateral loads imparted by seismic and wind load and the unbalanced soil load on the existing structure. In order to support the grade beam below the bottom of the excavation as well as provide resistance to lateral loading, micropiles were utilized. Micropiles also met the restrictions for tight access and low headroom with a clear span of less than 2.8 m (9.2 ft) on the existing lowest level.

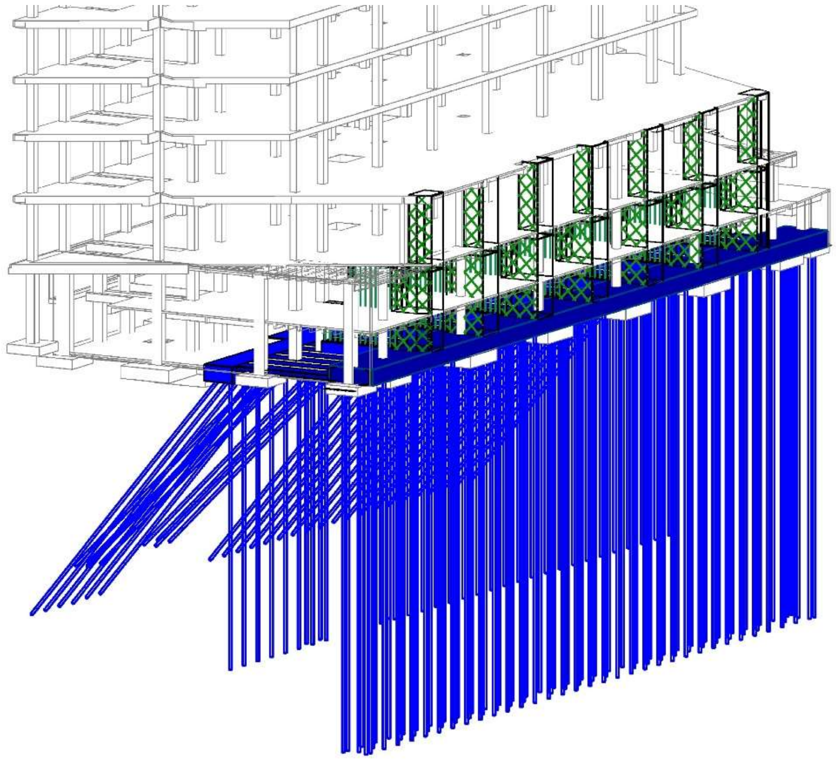

Figure 5. Selected Stabilization Option

FOUNDATION LAYOUT AND STIFFNESS

The maximum service loads were 3560 kN (800 kips) vertical and 1780 kN (400 kips) lateral at each new buttress. An initial approach was formulated between the geotechnical and structural engineers to install micropiles at 1.8 m (6 ft) on center along the length of the building cut plane to support the vertical services loads. This spacing was driven by the spacing of the soldier piles and tiebacks for the SOE. The micropiles along Column Line D were centered between the tiebacks to avoid a conflict.

A range of micropile diameters, with maximum capacities and spring constants, were provided to the structural engineer. This data was used to model the size of grade beam necessary to support the new lateral stability elements. The grade beam's stiffness, along with the micropile's spring constant, were used to determine both the vertical load distribution to the micropiles and the building settlement at the cut plane once the pit was installed. It was an iterative process. As the new grade beam got stiffer, the vertical loads were redistributed to the micropiles and the building settlement was altered. This change would adjust the length of micropiles required and their respective spring constant. After four iterations, it was determined that the grade beam's vertical loads would be supported on 95 vertical micropiles with a 427 kN (96 kip) capacity and a spring constant of 505 kips/in. A 178 mm (7 in) diameter casing with an 552 MPa (80 ksi) yield strength was used with a #14 epoxy coated center bar with a yield strength of 517 MPa (75 ksi). The casing extended for the full unbonded length (see Figure 6).

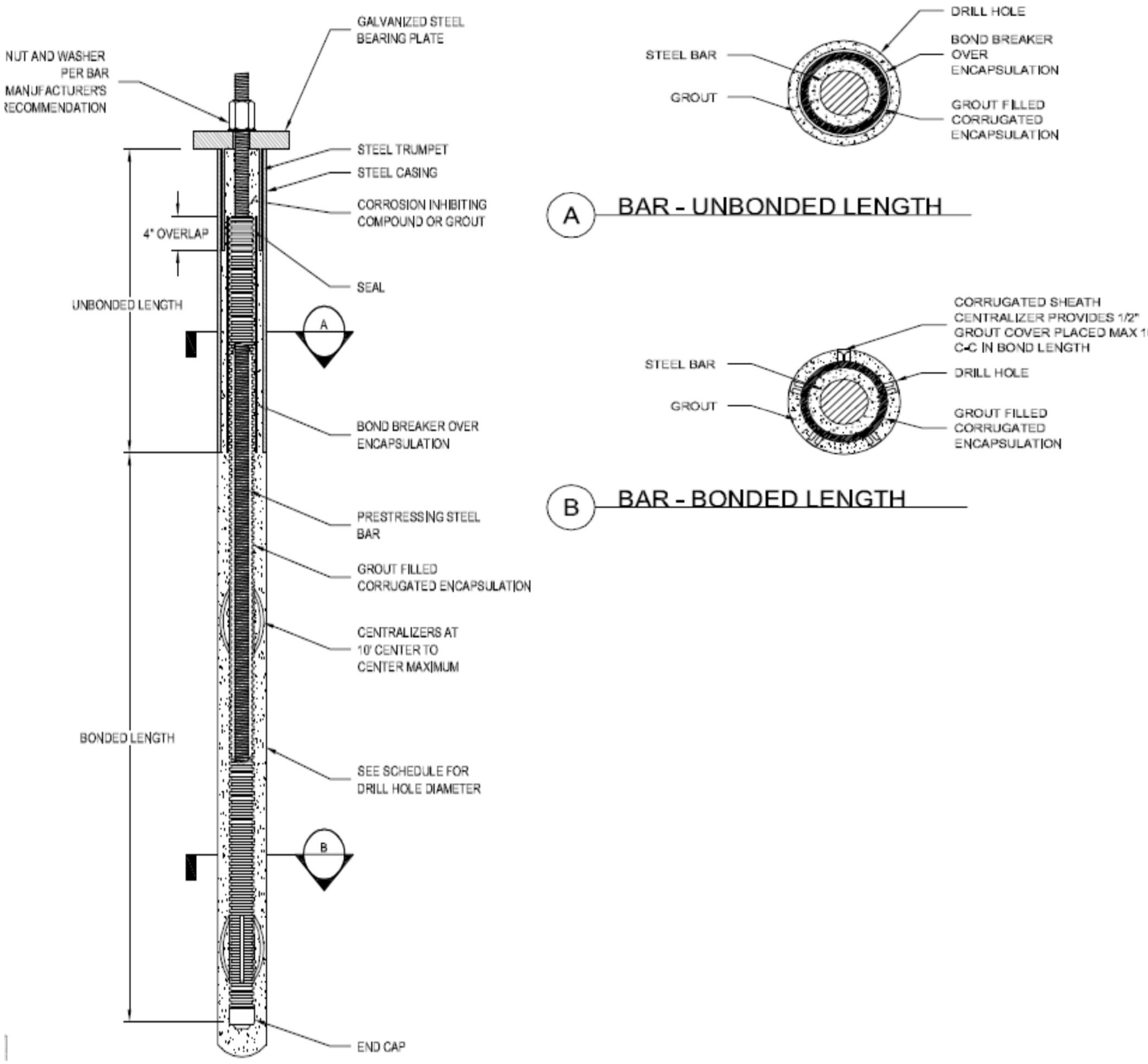

Figure 6. Micropile Details

This solution supported the vertical loads at Column Line D below the SOE, but did not address the lateral loading requirement. Ideally, battered micropiles would be installed through the new grade beam; however, the proximity of the SOE tiebacks precluded this option. Additionally, the battered piles would need to extend beyond the passive wedge formed by the 16.7 m (55 ft) deep bottom of excavation, making the battered piles much longer than would be needed for bonding into competent soils.

Installation of a post-tensioned battered pile coupled with a vertical micropile was determined to be the ideal solution. This A-frame system was located approximately

9.1 m (30 ft) away from the building cut plane and in line with the vertical micropiles along the front grade beam. A horizontal tie beam consisting of two post-tensioned tierods was used to transfer the lateral loads from the cut plane back to the line of A-frame micropiles (Figures 7 and 8).

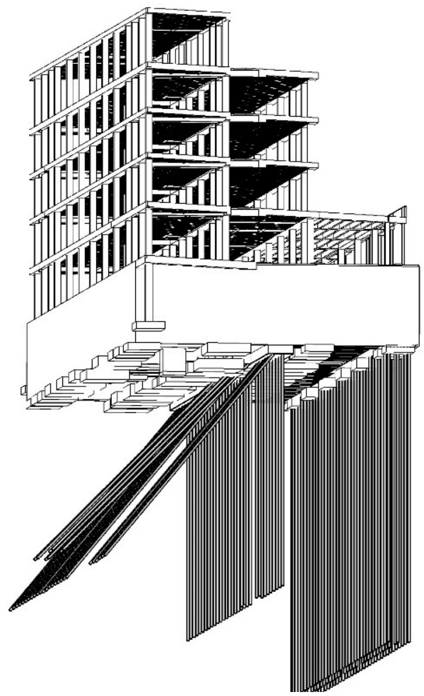

Figure 7. 3D View of Foundation Layout

In a similar fashion to the vertical load distribution, the stiffness of the new tie beam housing the A-frame piles was analyzed iteratively to determine how the lateral loads would be distributed to the coupled micropiles. As the grade beam size was adjusted for ideal load distribution, the coupled micropile capacities and spring constants were updated to reflect actual loads along each frame. Eventually, the ideal solution based on the coupled micropile lengths, battered pile installation angle for lateral load resistance, and stiffness of the front grade beam were determined. The final design included 34 coupled battered pile assemblies connected to 30 horizontal tie beams containing a total of 60 horizontal post-tensioned tierods.

The maximum horizontal load for most areas was 298 kN (67 kips) per 1.8 m (6 ft) width. Due to the inability to access and install micropiles around the existing elevator pit, the adjacent horizontal loads in those areas were 1014 kN (228 kips) per 1.8 m (6 ft) width. The design load of the vertical micropiles was 578 kN (130 kips) in compression, and the design load of the battered micropiles was 578 kN (130 kips) in tension (pre-stressed). The vertical piles consisted of a 178 mm (7 in) diameter, 552 MPa (80 ksi) steel casing with a #18 epoxy coated center bar with a yield strength of 517 MPa (75 ksi). The battered micropiles were designed as 44 mm (1.75) in All-Thread bar with a 1034 MPa (150 ksi) yield strength and Class I corrosion protection according to the Post-Tensioning Institute (PTI). Nicholson elected to use a four strand anchor rather than a solid bar for ease of tensioning.

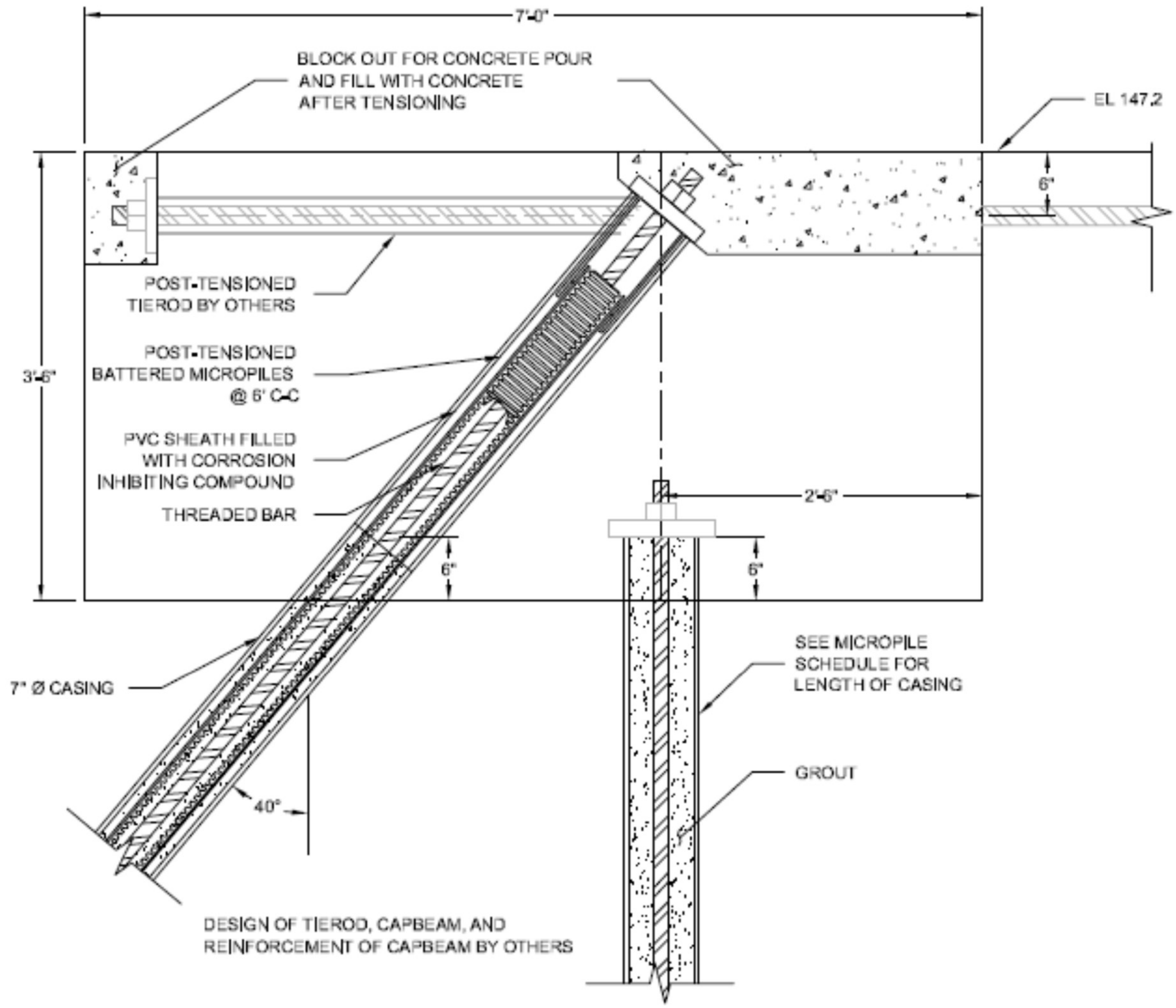

Figure 8. Battered and vertical micropile A-frame

INSTALLATION SEQUENCE

In order for the system to work as intended, an installation sequence was specified. All vertical and battered micropiles were installed before any new construction progressed. Once the micropiles were installed (see Figure 9), the new grade beams, shear walls, and buttresses were constructed. In addition, the horizontal post-tensioned ties were cast into the new tie beam and tensioned after the concrete had achieved the specified strength. This was done prior to advancement of the adjacent excavation in order to reduce movements. Lastly, the structural connections between the buttresses and shear walls were installed. Once this was complete the battered piles in the A-frame systems were tensioned and the portion of the structure to be demolished was removed.

Figure 9. Micropile Installation

MICROPILE BOND LENGTHS

The micropile bonds lengths and load transfer ratios were designed based on the results of tension load tests performed on four tiedowns installed from the Pavilion parking lot. The tiedowns were cased through the overburden soils and bonded 3 m (10 ft) into the desired stratum, whether terrace deposits, upper Miocene, lower Miocene, or a combination of upper and lower Miocene. A 140 mm (5.5 in) drag bit was used and the tiedowns were first gravity-grouted and then globally post-grouted in two stages using a sleeved-port pipe system. The test load was increased incrementally until geotechnical failure of the tiedown or until the maximum safe test load was reached.

Prior to installation of production piles, two sacrificial verification load tests were performed (TP39 and TP 58). Table 2 shows the mobilized load transfer ratio and bond strength in each bond stratum. An ultimate bond strength of 137 kPa (20 psi) was used for the design.

Table 2. Summary of Load Testing Results

Test Pile	Bond Stratum	Post Grouting Stages	Bond Length	Mobilized Load Transfer Ratio	Mobilized Bond Strength
1 (bar)	Terrace	2	3 m (10 ft)	267.1 kN/m (18.3 kip/ft)	552 kPa (80 psi)
2 (strand)	Upper Miocene	2	3 m (10 ft)	132.8 kN/m (9.1 kip/ft)	276 kPa (40 psi)
3 (bar)	Lower Miocene	2	3 m (10 ft)	105.1 kN/m (7.2 kip/ft)	220 kPa (32 psi)
4 (bar)	Lower Miocene	3	3 m (10 ft)	166.4 kN/m (11.4 kip/ft)	345 kPa (50 psi)
TP39 (bar)	Upper and Lower Miocene	2	11.9 m (39 ft)	109.5 kN/m (7.5 kip/ft)	228 kPa (33 psi)
TP58 (bar)	Upper and Lower Miocene	none	17.7 m (58 ft)	80.3 kN/m (5.5 kip/ft)	165 kPa (24 psi)

CONNECTION TO EXISTING FOOTINGS

Vertical micropiles installed along Column Line D were cored through the existing footings using a 228 mm (9 in) diameter hole that provided a 25.4 mm (1 in) annular space for the steel casing. The existing footings were 609 mm (24 in) deep and had a compressive strength of 28 MPa (4,000 psi). Using Figure 10 and an annular width of 25.4 mm (1 in), an ultimate bond stress of 3 MPa (430 psi) was conservatively used. Grout with an unconfined compressive strength of 34 MPa (5,000 psi) was specified.

CONCLUSIONS

This project was highly successful in multiple ways. Daily operations were maintained safely at the existing pavilion during construction. Instrumentation and monitoring data indicated that total horizontal movement of the existing building following construction of the SOE was approximately 10 mm (3/8 in).

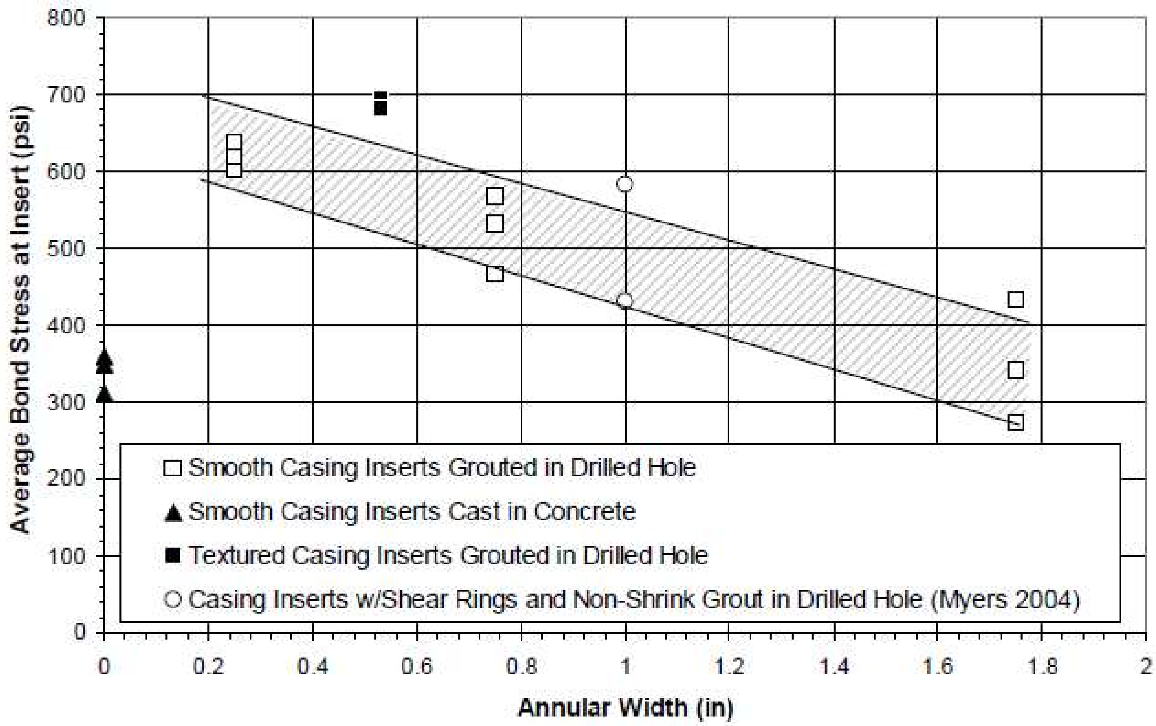

Figure 10. Bond Stress of Existing Concrete (From Gomez et al, 2005)

The project successfully consolidated many pediatric services into one building to provide comprehensive outpatient care under a single roof. This $200 million investment by VCU Health also improved the historic and vital Broad Street corridor of Downtown Richmond. Sustainability in construction and design was a focus of the project. The creative engineering solutions by Schnabel and DMWPV allowed effective redevelopment of an existing brownfield site rather than building on a Greenfield site. VCU Health is seeking LEED Silver certification from the U.S. Green Building Council.

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