Use of Micropiles to Rehabilitate High Rise Tower Foundations in Vancouver, B.C.

Matt Kokan, M.A.Sc., P.Eng., GeoPacific Consultants Ltd.

Abstract

A 33 storey high rise tower in downtown Vancouver, Canada experienced flooding in the lowest parking level was experiencing flooding resulting in a maintenance call to the building contractor. The writers company was geotechnical engineer of record for the project, that had been completed 2 years prior and were asked to review the site. A site review was conducted after sufficient de-watering had taken place revealing that a large cavity had developed adjacent to the tower core and several tower column foundations. Emergency measures were carried out to fill the cavity. This paper discusses the investigation of the cavity as well as conditions around the cavity and the design and implementation of a repair strategy that was ultimately used to reinstate support for the foundations that had their support compromised. Significant technical expertise and practical support was provided by the contractor who provided an alternate methodology for completion of the works and ultimately this strategy proved to be beneficial to the repair work.

1.0 Introduction

A 33 storey high rise tower was completed in 2005 in downtown Vancouver. Five levels of below grade parking were required for the project so that foundations were constructed up to 18 m below surrounding street grades. Several years after the tower was occupied the writer was contacted to review a drainage problem reported by the property management group. A review of the drainage sumps at the lowest level parkade sump showed that a large void had developed below the tower, partially undermining the core foundation and a number of the tower column foundations. Coincidentally a transit tunnel had been constructed, with the crown of the tunnel located approximately 15 m below foundation elevation.

This report describes the options reviewed for foundation repairs and describes the eventual repair methodology completed to reinstate foundation support using micropile foundations installed with a staged grouting procedure to provide adequate load transfer to foundation soils but also fill voids that were anticipated could exist below the foundations.

2.0 Project Description and Subsurface Conditions

GeoPacific Consultants was retained to provide geotechnical engineering services for the project in about September 1999. The project consisted of two high rise towers connected to a low rise podium structure all over a split 4/5 below grade parkade. The west tower, which is the subject of this paper, rose 33 stories above street level and was situated over top of the split 4/5 level parkade. The parkade below the west tower was constructed up to 16.7 m below street grades around the site. Since the elevators had to service the 5th parkade level, the deepest portion of the structure was in exces of 18 m below street grades. The building was entirely constructed of reinforced concrete. Superstructure loads were heavy, in the range of 25,000 kN for columns and in excess of 200,000 kN at the tower core. The excavation was completed in about June of 2003 and so foundations were constructed about this time. Foundations were conventional pad and strip footings with a mat foundation for the tower cores.

The geotechnical investigation had anticipated dense to very dense lodgement till overlying very dense pre-glacial sand. The sand was fine grained and silty and contained zones of sandy silt, with fines contents (percent passing the 75 micron sieve size) ranging from 25 to 65 percent. Standard Penetration Test values ranged from 50 to 100. The groundwater level was located at about 14.5 m below street grades, which was about 3 metres above the deepest excavation grades. Groundwater management during construction was a significant effort as was excavation support in the final 2 to 3 m.

A design section of the shoring attached as Figure 1, shows the depth of the parkade excavation below street grades.

3.0 Problem Recognition

In the late afternoon of April 5, 2007, the writer was enjoying a refreshment at a local hotel lounge in downtown Vancouver, when the phone rang. It was the superintendent for the contractor that constructed the subject tower. He said that he got a phone call from the strata managers who reported that the P5 level of the parkade was flooding. I suggested that the pumps for the drainage sump had likely failed which was agreed to by the superintendent. The superintendent visited the site to review the problem. About 30 minutes later the phone rang again. It was the superintendent again, who reported that the sump pump was gone and that there was a large hole visible below the lowest level floor slab. He asked the writer to come down to the parkade and meet him and the strata manager. The parkade was dimly lit and there was a 100 mm diameter flexible hose exiting from a circular hole the in the P5 slab where the lid for the sump pump was previously located. I got down onto my stomach with a flash light and looked beneath the slab. There was a void about 20 m by 15 m across. I could see the core footing partially undermined as well as a number of tower column footings with nothing but water beneath them. The water was clear and provide a good view of the entire void. The depth of the void was measured with at tape at 5.8 m, at its deepest point. After a few minutes I realized that I was lying on a 100 mm thick unreinforced concrete slab that was spanning up to 20 m with a water depth of up to 3 m below me. I moved away quickly and asked the others that were in the P5 level to do the same. The project structural engineer and a concrete form work contractor were called immediately to review the condition of the structure.

4.0 Response to Problem

4.1 Initial Response

In the following 2 to 3 days approximately 165 m3 of high strength flowable concrete was pumped into the void until the edge of the foundations were no longer visible. 24 hours after the concrete was pumped in the slab was shored and the mechanical and electrical systems were reinstated so that the building mechanical and electrical systems were fully functional. Flowable low strength concrete was then placed to the underside of floor slab elevation to support the slab.

It was anticipated that a significant zone of disturbed sand existed beneath the high rise tower and that this would need further remediation to reinstate the pre-disturbance safety of the building.

During the period between the time the void was recognized and the initial response was completed it was determined that a tunnel has been advanced beneath this tower for the Vancouver's new north south metro line connection to the airport in Richmond, B.C. This was part of a commitment based for the Vancouver Whistler 2010 Olympic bid. Only the outbound tunnel was completed by this time and the parallel inbound tunnel had yet to be bored. It was to pass beneath the sister (north) tower according to plans.

Discussions took place with the tunneling group that ultimately led to a detailed review of subsurface condition beneath the outbound tunnel bore, including soil conditions and building monitoring. The second bore was completed without incident. The paper will present data collected from the investigation however the second bore will not be discussed in this paper.

4.2 Final Support Design Development

The building strata (residents association) requested a meeting early after the initial repair had been completed to have us explain why the P5 level at the south tower was blocked off. The writer advised the strata members of what had taken place and naturally they were horrified and concerned regarding the safety of the building. A review of the structure at the time the problem was recognized, showed very little if any structural distress and no indications that foundations had moved significantly. This was a surprise to the writer, but given the manner in which modern reinforced concrete buildings are constructed and in particular the extent of reinforcing that must be added to achieve seismic performance in Vancouver's high seismic environment maybe it should have been expected.

In our subsequent meetings with the building strata group and the property managers it became clear that the strata had negotiated an easement beneath the building for the purpose of boring a tunnel and had received monies from the group that was constructing the tunnel. The strata had requested that we provide them with certification that the building was now safe to occupy and that all necessary repairs were complete. We advised them that in our opinion further work was required to reinstate foundation support to the level that existed prior to the tunneling. We had a number of meetings with the strata and the tunneling group which were aimed at developing a mutually agreeable solution and of course the obvious question of cost and who was paying were hot issues, but not discussed directly at our joint meetings. In separate meetings with the strata we had advised that an investigation to determine the extent of subgrade softening was required to develop a proper repair strategy. Our initial thought was to use jet grouting to complete the repair works. The tunneling group was opposed to this approach as they felt it could negatively impact the tunnel as a result of stress concentrations near the tunnel roof due to the difference in stiffness of the soil/cement compared with the in situ soils. We felt that not withstanding these concerns, jet grouting would best reinstate the stiffness and strength of the subgrade soils prior to disturbance and had the advantage of not requiring any physical attachment to the existing foundations.

An investigation program was developed to study the extend of disturbance beneath the south tower. Since the headroom at the P5 level was only 2.5 m, a mini drill was mobilized to complete the investigation. A Titan hollow bar was used to penetrate the lean concrete and high strength concrete fill that had been previously placed and then probe the subgrade below. The disturbed zone was readily identifiable by permitting penetration of the hollow bar anchor with out any downward pressure, advancing the hollow bar with rotation only. The results of the drilling showed an extensive zone of loose sand (disturbed natural subgrade) remained beneath the concrete. The average depth was about 6 to 8 m, decreasing significantly away from the centre of the disturbance zone. The zone of disturbance is shown in plan and section on the attached Figures 2 and 3. Fortunately the centre of disturbance was not located beneath any of the tower columns or the core footing.

An initial design and specifications was developed for the jet grout option and presented to the building strata for review. Budget pricing was requested from the local contractors to determine the likely repair costs. Initial feedback suggested that the repair work cost for this option was on the order of $1 million Canadian Dollars. The strata discussed this option with the tunneling group who rejected it as a solution and offered their own repair option which was reticulated micropiles (Type II), centred on the disturbance zone. Hollow bars were proposed for the reticulated elements. We had concerns regarding this approach and to avoid a long protracted argument we suggested to the strata that they seek an independent third party engineering opinion on the two approaches (direct support using jet grouting vs indirect support using type 2 micropiles). The third party was not comfortable with the indirect support methodology proposed and supported our design. Ultimately it did not matter since direct meetings between the strata and their legal advisors and the tunneling group suggested that the repair costs could not be supported by either group. Insurers were involved at this point as the strata had notified them and they advised that the strata insurance for the south tower would increase by $110,000 per year until the repair was completed. Now the strata was rather motivated to complete the repairs. The feed back given to us was that a repair cost of on the order of $250,000 would be acceptable to the strata and the tunneling group.

Our challenge was to provide an alternate strategy for repair that achieved the necessary building safety within the allocated budget. After some internal meetings and discussions, it was determined that drilled hollow bars with high pressure grouting during installation could likely be an economical solution, since it would ensure that any voids beneath foundations would be filled and ensure load transfer can occur into undisturbed soil using a combination of the grout, grout improved soil and the hollow bar structural capacity. Discussion with the structural engineers confirmed that connection to the existing foundations could be based on concrete/grout adhesion alone and no special connections were to be required.

5.0 Final Design Development

Design drawings and specifications were prepared by GeoPacific based on the concept described above. The the repair involved removal of slab on grade around the edge of foundations at all locations of repair, followed by exposure of the top of foundation elements. Once the foundation elements were exposed the micropiles would be drilled through the foundations to provide support and permit grouting under pressure and consequent ground improvement in zones of disturbance. The proposed hollow bar micropile layout proposed by GeoPacific is shown on the attached drawings, Figures 4 and 5.

The work was tendered and pricing came back with a significant disparity in the high and low bids. The low bid, Kani Foundation Technologies, provided an alternate design. The design, shown on Figure 5 and 6, attached, called for drilled and grouted solid bar, with secondary grouting proposed at 4 levels. The post grout tubes were to be positioned strategically at depths where drilling (for primary grouting) observed soft zones or layers. This required the contractor to assemble the bar, with centralizer and post grout tubes on site, with every installation customized to the ground conditions. A general layout of the micropile underpinning elements with respect to the foundations, the micropile lengths and general installation parameters are shown on Figure 6 and 7. A total of 23 grouted micropiles were specified for the final design, with a length that varied from 7.9 m to 9.1 m.

6.0 Execution of the Design

Kani Fondation Technologies mobilized an in house built mini drill rig with a 2.2 m length, 0.7 m width and 2.2 m mast length to the site to complete the scope of work. The drill weighs 1,200 kg and produce a peak torque of 4000 N-m. Prior to drilling, the concrete floor slab was saw cut and removed in the foundations areas where the micropile elements had been specified. A 178 mm core was advanced through the foundations and high strength concrete at each micropile location. A nominal 150 mm casing was used to develop the micropile length. Drilling effort was recorded to track soft zones where post gout tubes were to be installed along the with threaded bar and centralizers. Generally the number of post grout tubes varied from 2 to 4 and the minimum separation used was 600 mm. Primary grouting was done as the casing was withdrawn at a grouting pressure of 8.5 bar. If more than 2 zones of soft soil were observed during drilling, 50 mm plastic tubes were installed at the soft zones, without threaded bar, rather than post grout tubes attached to the threaded bar. Grout was mixed with an Acker colloidal grout plant with pressure control valves for grout mixing and pumping.

Post grouting was done at between 18 and 24 hours after primary grouting. Pressure used for secondary grouting was 51 bar. Where the secondary grouting was done prior to thread bar placement, the location was re-drilled the following day (18 to 24 hours after secondary grouting) and drilling observation was used to determine where tertiary post grouting was to be completed. This procedure was suggested by the conractor (Kani) after drilling commenced and they recognized the extent of disturbed materials that surrounded the drill holes. As the designer and engineer of record I really appreciated the contribution of technical expertise that Kani provided to this project. It would not have been anywhere near as good a result if we had proceeded with the initial design or the design proposed by the tunneling group.

The volume of grout was carefully monitored using a Tigermag digital flow meter during initial (primary) grouting, secondary grouting, and where undertaken, tertiary grouting. In addition monitoring of the floor and building was continued to ensure that pressure was not causing undesirable lifting of the slab or building superstructure.

The contract assumed 400 bags of grout would be used and the contractor was to be paid extra where this amount of grout was exceed. A bag of grout weighs 30 kg and yields about 0.018 m3 when mixed to the recommended water cement ratio. Thus about 56 bags of grout would yield 1 m3 of in place grout. Since the subgrade material was sandy, we did not anticipate actual voids in any of the holes, nor did we observe any in the drilling, though there were numerous zones where very loose soil was identified. As designers we felt that 7.1 m3 (400 bags) would be more than necessary to complete the 23 micropiles (about 0.3 m3/pile).

Reference to Figure 8 shows the results of the grouting recorded during micropile installation. Primary grouting (done at 8.5 bar) consumed 346 bags or 6.2 m3 of grout. Based on a total installed length of 206 m of micropile with a theoretical hole diameter of 150 mm, a minimum grout take of 3.6 m3 was expected. Thus the actual grout take was 72 percent over the theoretical minimum. Secondary and tertiary grouting consumed a further 9.7 m3 of grout (543 bags). The distribution of grout take during secondary and tertiary grouting is shown on Figure 8, with the magnitude of secondary grout take represented graphically by the diameter of the circle. Not surprisingly locations with larger primary grout take had larger secondary and tertiary grout take. One of the micropile locations (A5 on Figure 8) accounted for 47 percent of the grout take (7.9 m3 or 422 bags). This location, between the core footing and one of the tower columns, was assumed to be the centre of the disturbance cone, with the cone likely directed down towards the crown of the tunnel.

Overall we were surprised by the amount of grout consumed during the micropile installation however we were also confident at the end that we were correct in assuming that the initial (emergency) response would not be sufficient to ensure long term safety of the tower.

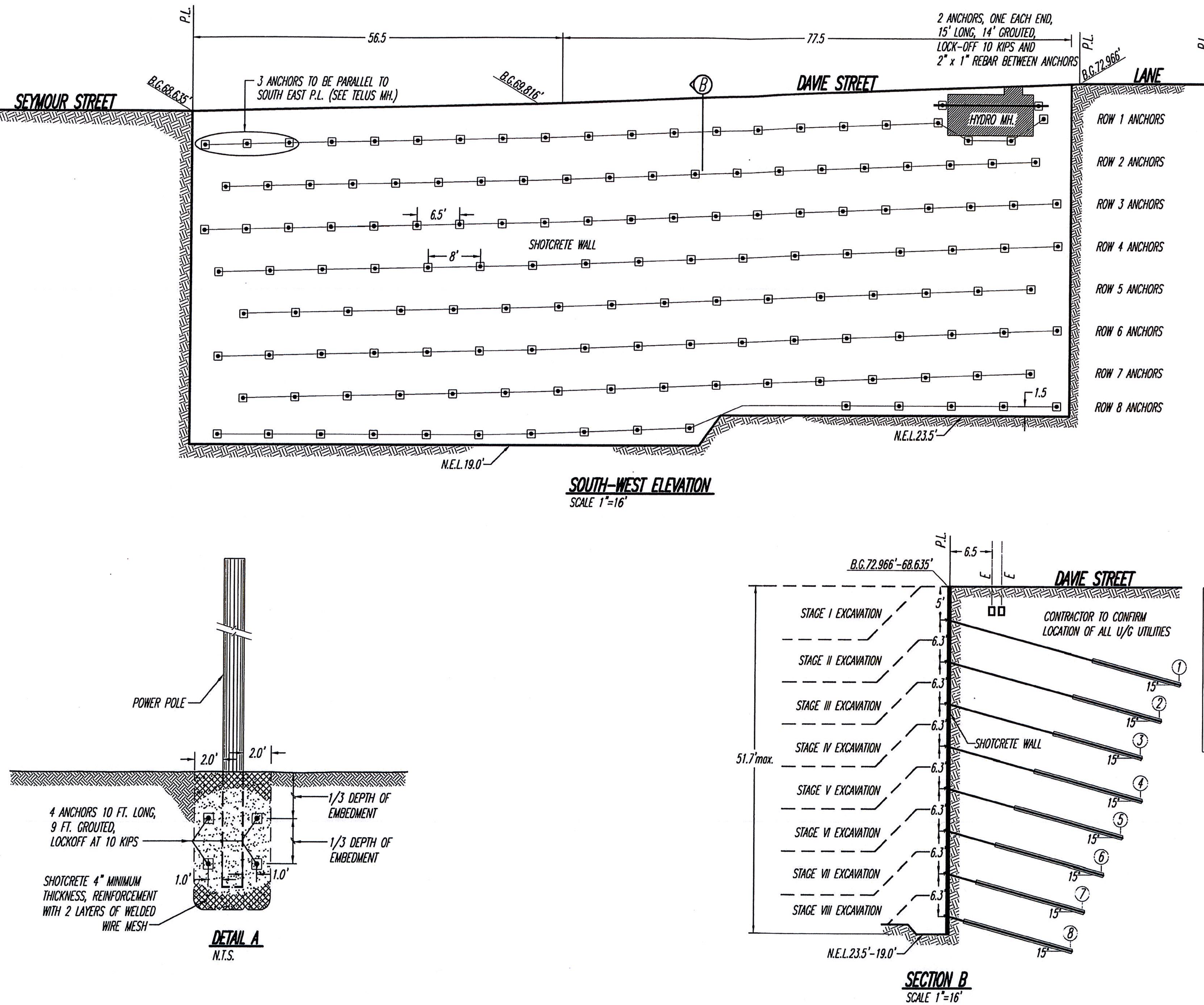
6.0 Conclusion and Acknowledgements

A micropile based solution was used effectively to support a very heavy structure that was originally intended to be supported on conventional foundations placed on a prepared subgrade of dense pre-glacial sand. The repair solution had to recognize the need to not alter the original design intend, address the challenges in head room and other access difficulties associated with the work place and ensure safety of the 33 storey tower filled with residents. Micropiles offered a very flexible yet robust solution to a complex problem and the monitoring and quality control provided by both the engineer and the contractor provide confidence of a repair that is well done.

It is not clear where all the grout (16 m3) went and what effect was actually occurring during grouting, compaction, cementation, mixing or micro fracturing. We do know the slab was not lifted so major ground fracture was not a mechanism responsible for the majority of the grout consumption. There are probably a number of effects that in combination led to the success of the repair.

From past symposiums, talks and technical presentations, It is clear to the writer that it is the contractors and suppliers and not the designers or the academics that are leading advancements in the field of micropile use. I expect that this will continue as they have the practical experience, technical expertise and desire to refine and advance their methods.

I would like to acknowledge the valuable assistance of Kevin Liu of GeoPacific and William Auyeung, P.Eg and Anthony Tam, P.Eng., of Kani Foundation Technologies Inc. in providing technical support and assistance for this paper.



DYWIDAG THREADBARS OR APPROVED ALTERNATE

ROW	LENGTH (ft)	GROUTED (ft)	LOCKOFF (kips)	SPACING (ft)	
1	37	14	41	6.5	#8 Gr. 75/100
2	34	14	41	6.5	
3	31	14	41	6.5	
4	31	17	50	8	#9 Gr. 75/100
5	28	17	50	8	
6	25	17	50	8	
7	22	17	50	8	
8	20	17	50	8	

Figure 1. A Design Section - Southwest Elevation of The Shoring

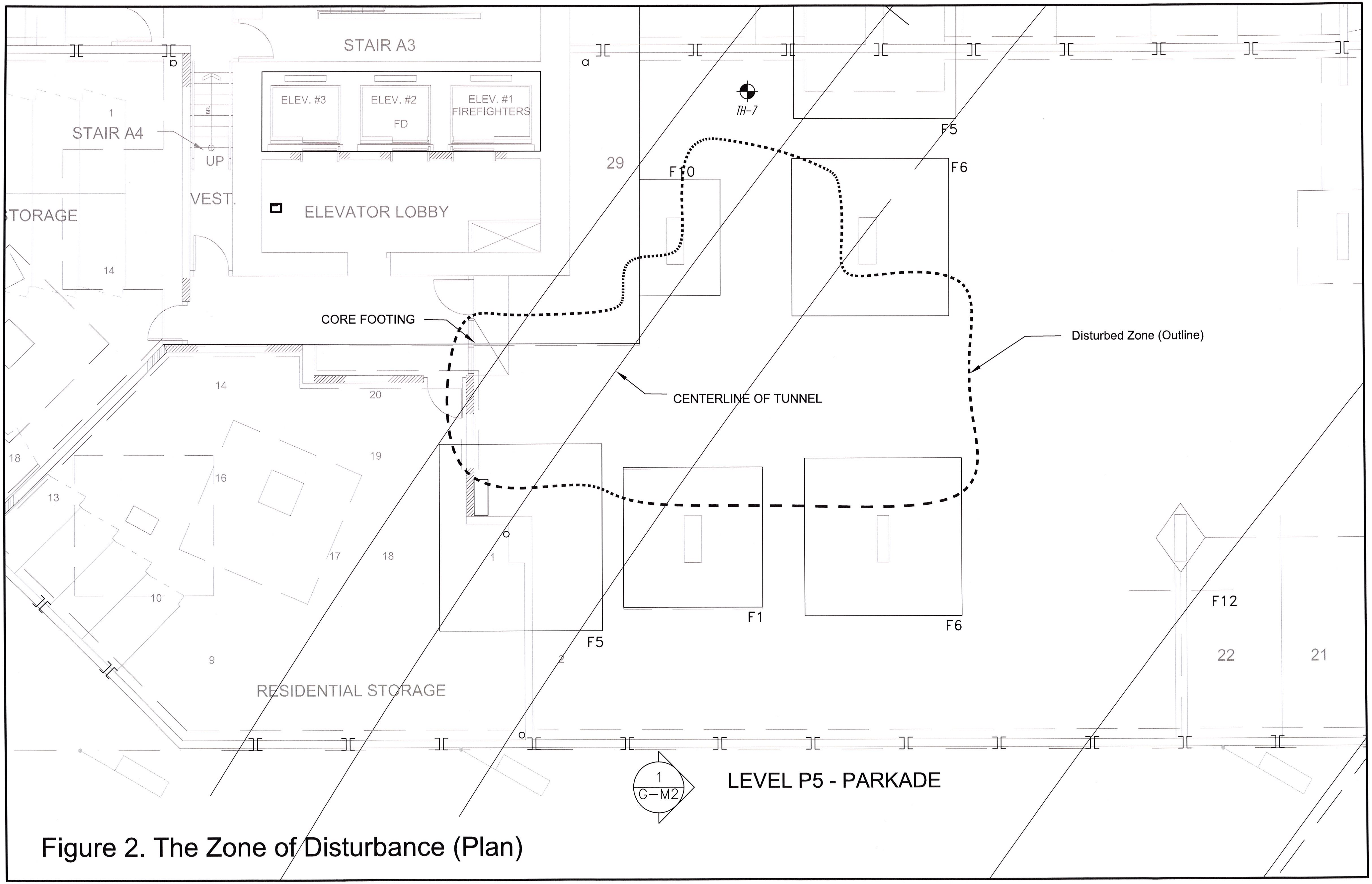


Figure 2. The Zone of Disturbance (Plan)

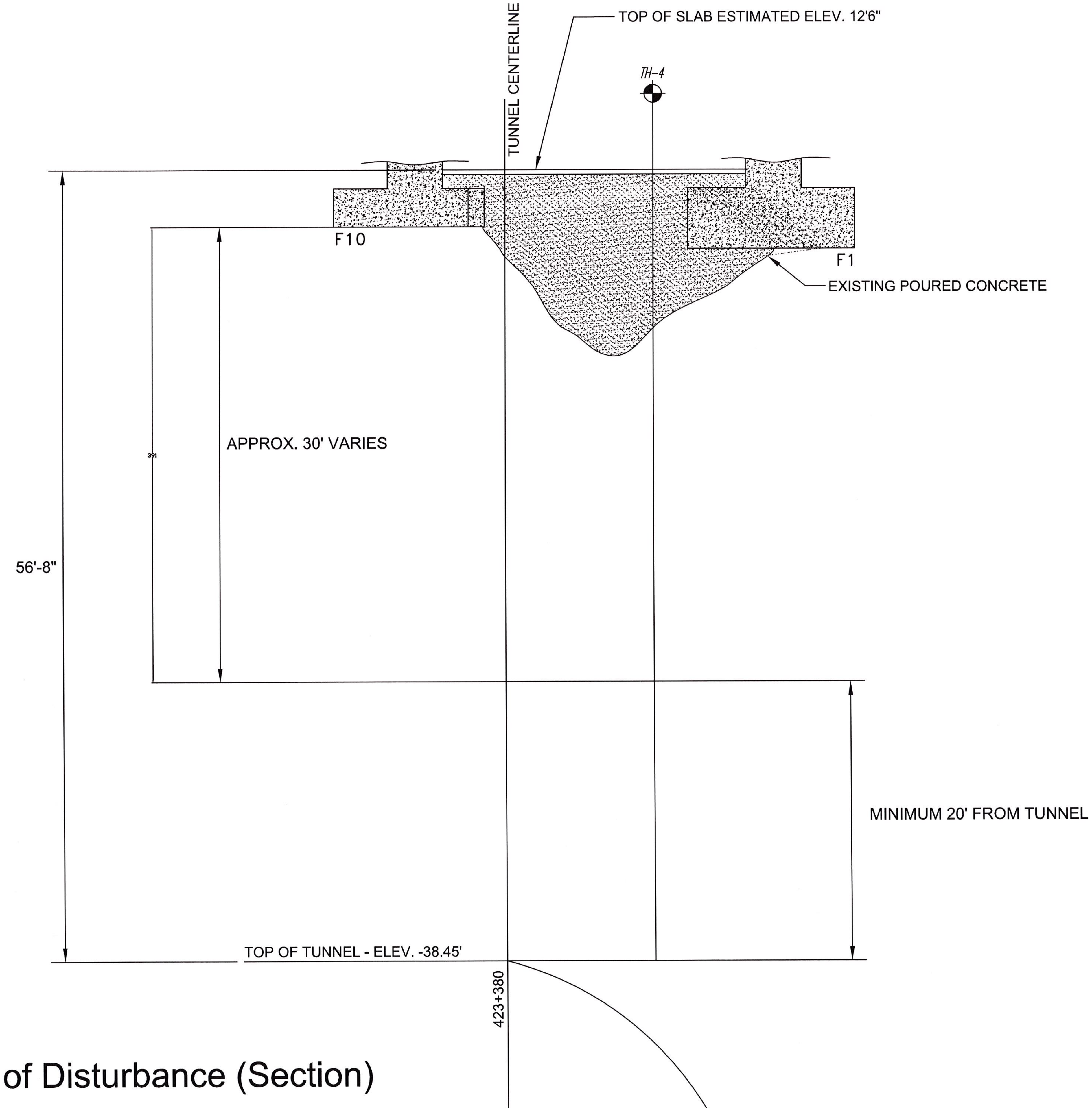


Figure 3. The Zone of Disturbance (Section)

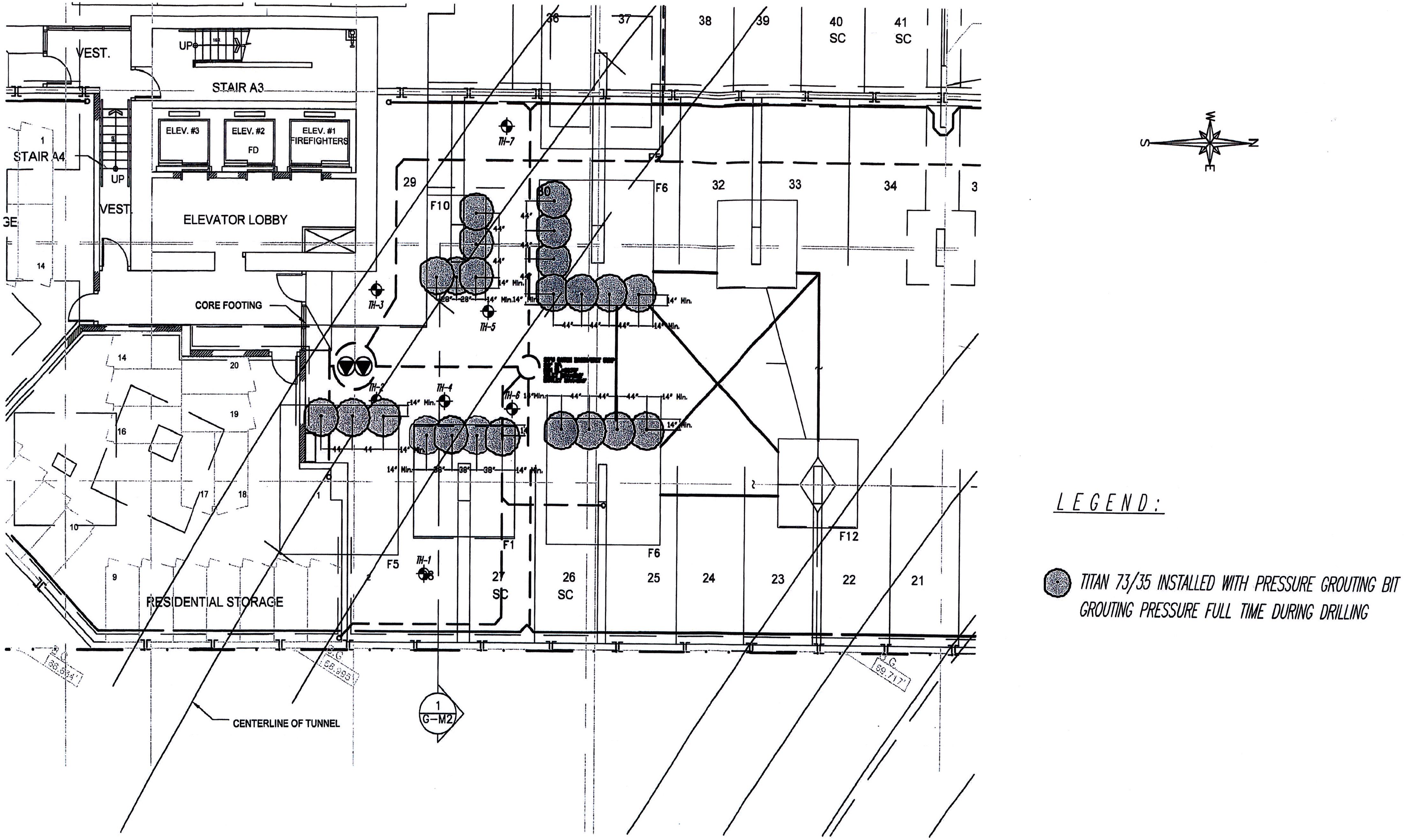
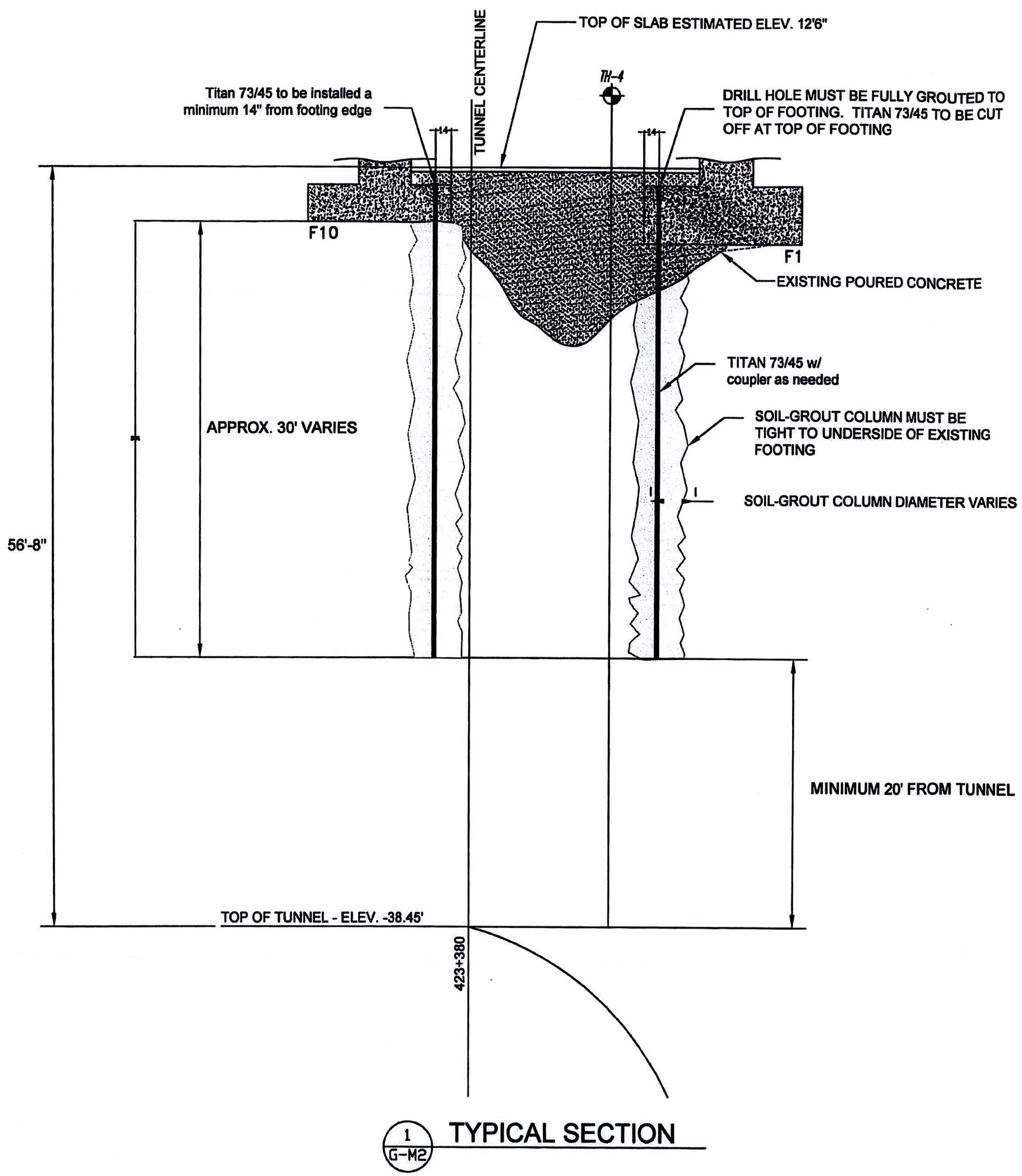


Figure 4. The Proposed Hollow Bar Micropile Layout Proposed By GeoPacific Consultants Ltd.



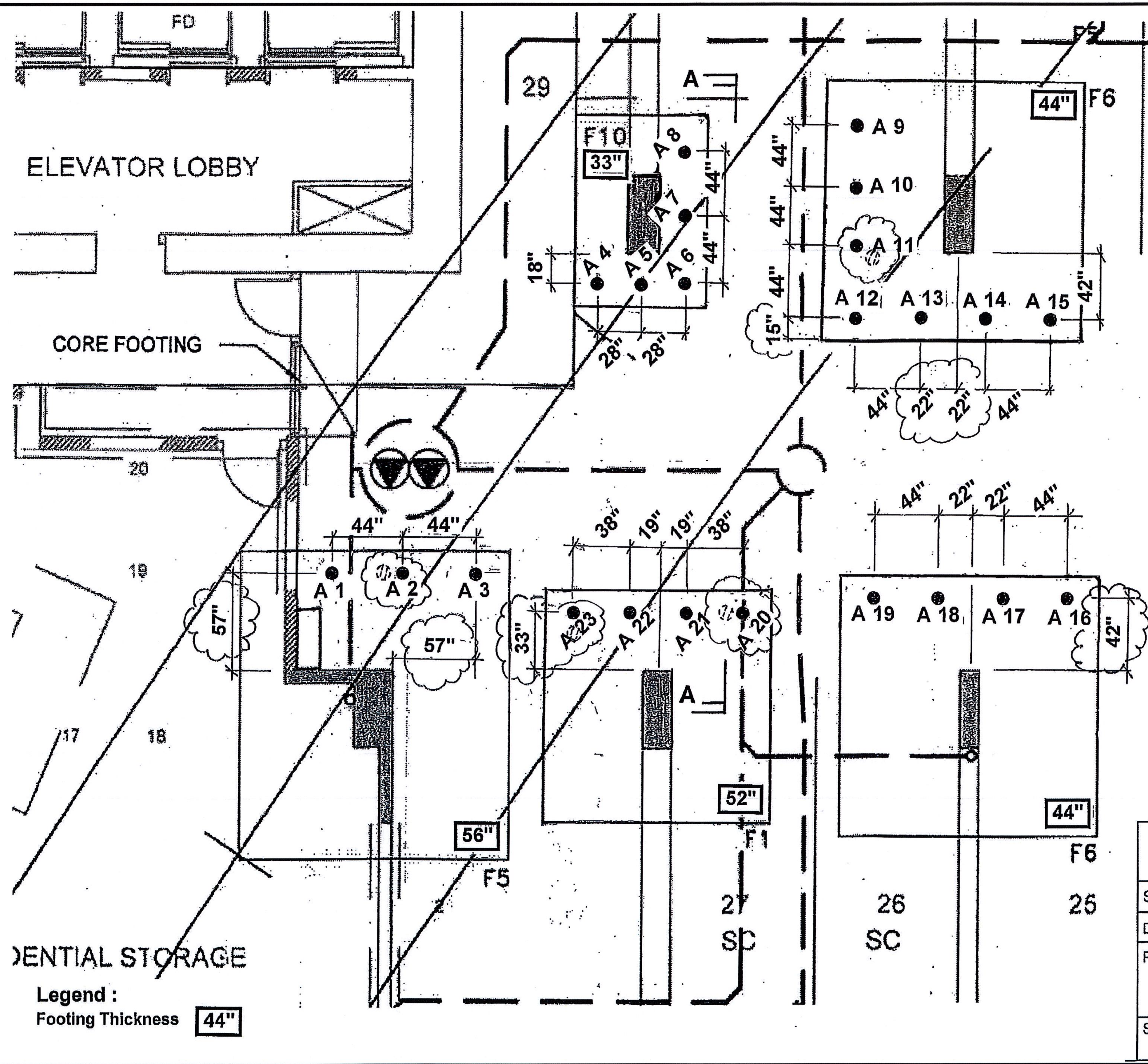
GENERAL NOTES:

1) Scope of work:
This work includes furnishing Micropiles with a soil-grout column mixture for load bearing under pad footings. The micropile will consist of drilling a hollow bar Titan 73/45 with a dynamic jetting system, to create a soil-grout mixture column surrounding the bar.

2) Micropile Installation Materials and Equipment
-Titan Micropile (hollow bar 73/45) shall conform to ASTM 615, grade 75.
-Use only Titan couplers to splice micropiles
-Drilling eqiupment shall be hydraulic rotary or rotary percussion with top hammer
-Grout mixer shall be high speed colloidal mixer with seperate holding tank and water/cement dosing system to ensure continuous grouting independant from mixing.
-Grout pump with at least 120L/min volume and minimum pressure of 1200psi (80 Bar).
-Drill bits to be suitable for the ground conditions with threaded jet inserts.
-Grout mix should consist of Portland cement Type I, II or III, mixed with a water/cement ratio (W/C) = 0.8-1.0
-Grouting shall be full time during harder ground resistance, for lower resistance drilling grouting pressure should be reduced and drilling bar retracted and advanced as needed

3) Design
The micropile layout shall be in accordance with the design developed and sealed by GeoPacific Consultants Ltd.

Figure 5. Design of The Micropile



Anchor Grouted Pile Table

Anchor Grouted Pile No.	Existing Footing			Anchor Length
	No.	Size	Thickness	
A1 - A3	F5	14' X 16'	56"	26 ft
A4 - A8	F10	7' X 10'	33"	26 ft
A9 - A12	F6	13.5' X 13.5'	44"	28 ft
A13 - A15	F6	13.5' X 13.5'	44"	30 ft
A16 - A19	F6	13.5' X 13.5'	44"	30 ft
A20	F1	12' X 12'	52"	30 ft
A21 - A23	F1	12' X 12'	52"	28 ft

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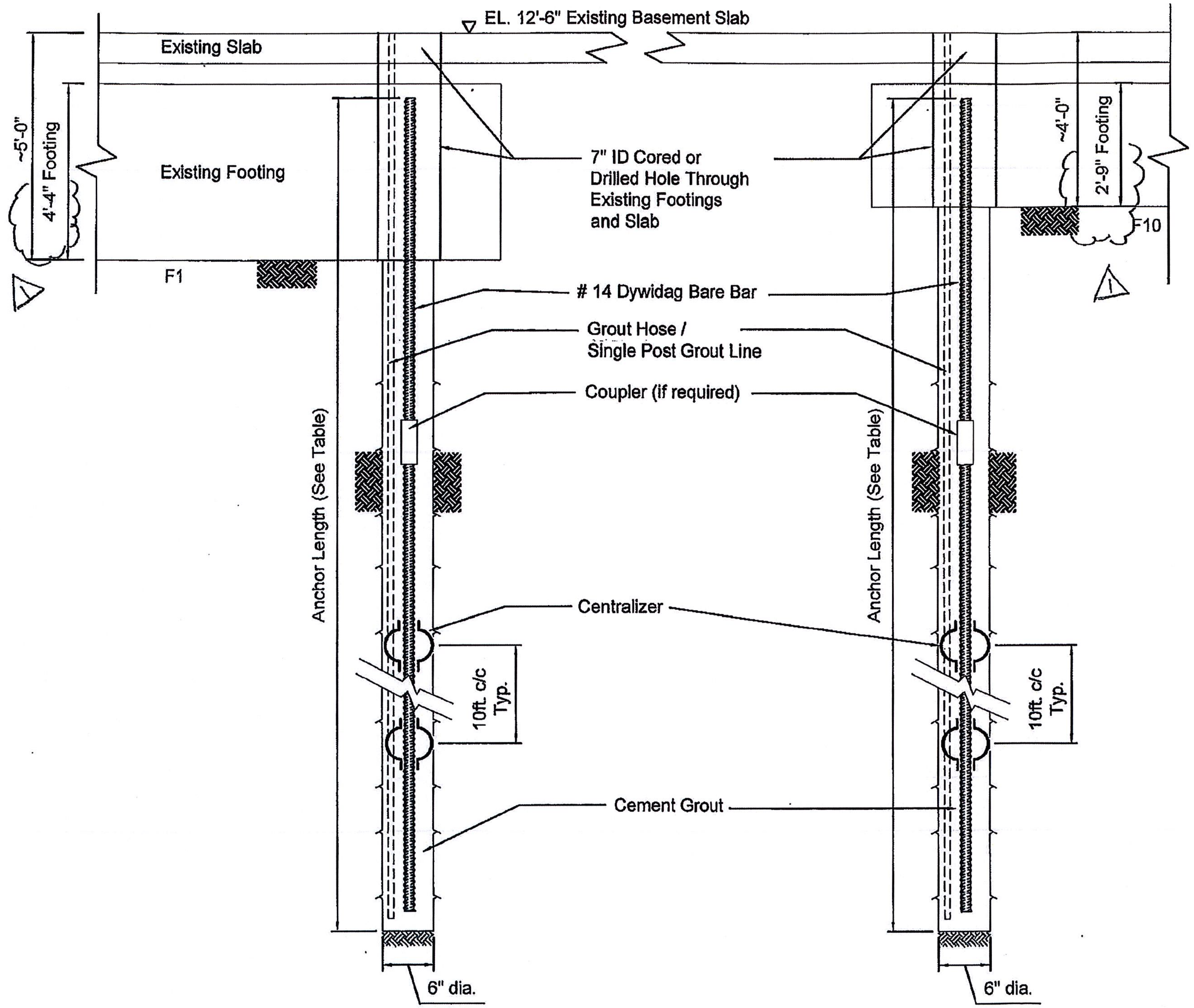
SCALE: AS SHOWN	APPROVED: AT	DRAWN BY: TT
DATE: SEPT. 23, 2010		

Project: 1199 Seymour Street, Vancouver

Sheet title: ANCHOR GROUTED PILES LAYOUT PLAN

DWG.No: WA2010-18-02 Rev.01

Figure 6. General Layout of The Micropile Underpinning Elements With Respect To The Foundations



A Typical Anchor Grouted Pile Details

General Notes.

1. Material
   1.1 Anchors
   Grade 75 Ksi Dywidag #14 Threadbar by Dywidag Systems International.

   Yield Capacity of Threadbar = 1334 kN
   Ultimate Capacity of Threadbar = 1779 kN

   1.2 Grout
   Type 30 Cement, w/c = 0.5(max) for primary grouting with maximum pressure = 125psi.
   Type 30 Cement, w/c = 0.7(max) for post-grouting with maximum pressure = 800psi

2. Drilling
   Drill hole by using air percussion drilling method with or without casing liners as required.

3. Installation and Grouting
   3.1 Install the pre-assembled threadbar into the bore hole.
   3.2 Attach a single post grout line to the anchor with the valve placed at the relatively soft soil strata determined during drilling operation
   3.3 Trimie grout the bore hole with the grout line attached to the bottom of the Anchor.
   3.4 Post grout the anchors to maximum 800psi or 6 bags of Type 30 cement whichever reaches first.

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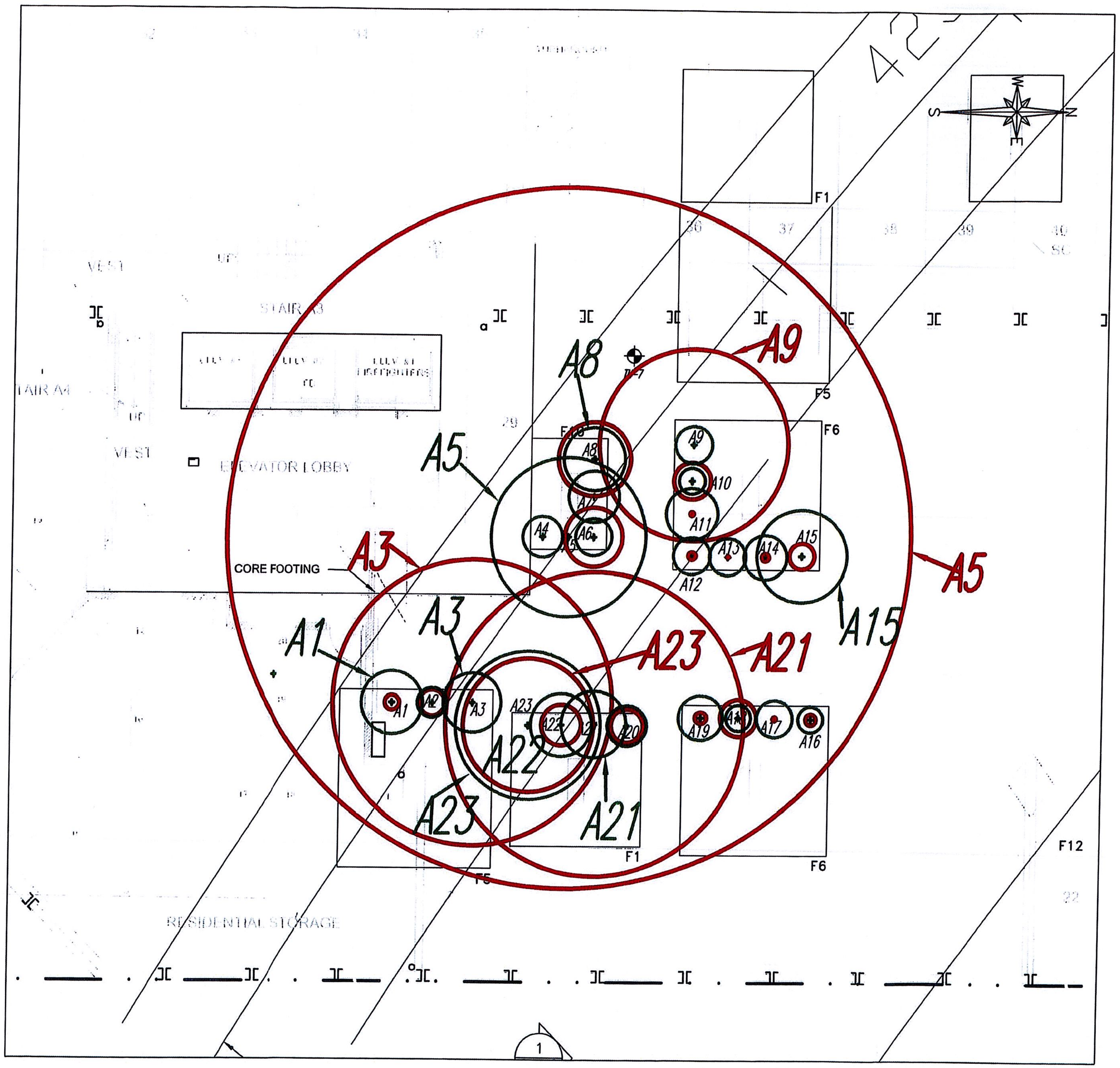
DATE: SEPT 23, 2010		
Project: 1199 Seymour Street, Vancouver		
Sheet title: PROPOSED ANCHOR GROUTED PILE NOTES AND DETAILS		DWG.No: WA2010-18--01 Rev.01

Figure 7. Proposed Micropile Notes And Details



Summary of Drilling and Grouting at Brava Site

Location	Drilled Depth [ft]	Primary Grouting [bag]	Post Grouting [bag]	Bags of Cement [bag]
A1	26	17	4	21
A2	26	8	7	15
A3	26	16	77	93
A4	28	11	NA	11
A5	28	43	189.5	232.5
A6	28	10	16	26
A7	28	14	NA	14
A8	28	17	20	37
A9	30	10	52	62
A10	30	7	10	17
A11	30	14	1	15
A12	30	10	2	12
A13	31	10	0.5	10.5
A14	31	12	2	14
A15	31	25	7	32
A16	31	7	3	10
A17	31	10	1	11
A18	31	7	10	17
A19	31	12	3	15
A20	31	11	9	20
A21	30	18	82	100
A22	30	17	11	28
A23	30	40	36	76
In total	**676**	**346**	**543**	**889**

LEGEND:

GREEN CIRCLE – DENOTES SCALE OF PRIMARY GROUTING

RED CIRCLE – DENOTES SCALE OF POST GROUTING

Figure 8. Results of Drilling And Grouting of The Micropile Installation