DESIGN-BUILD SUPPORT OF EXCAVATION FOR BLOCK 76

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The Block 76 project includes the demolition and redevelopment of an entire city block for mixed use retail, office and residential properties directly adjacent to Historic Temple Square in Salt Lake City, Utah. Excavation was carried out 65 ft below street grade and by as much as 50 ft directly below adjacent buildings supported by shallow foundations. Movements of the earth retention system and adjacent buildings were monitored in real time using automated equipment which complemented traditional geotechnical instrumentation. In the areas immediately adjacent to the existing buildings 9 stories or greater, anchored diaphragm walls were employed. Areas of the site not supported by diaphragm walls employed a soil nail wall system. Jet grouting was used for water cutoff in lieu of dewatering along the north and east sides of project.

In total, the earth retention scope included 45,000 sq. ft of diaphragm wall, 104,000 sq. ft of soil nail wall, and micropile underpinning of 3 buildings. This paper provides an overall description of the Block 76 project. A detailed account is provided of the technical and commercial lessons learned. The project provided a unique example of how an owner can receive best overall value by acceptance of some technical risk up front. Benefit was gained by allowing the contractor to reduce contingencies within the bid from unknowns on a fast-track project.

BACKGROUND

City Creek Center is a US$1 billion project in downtown Salt Lake City, Utah. The project is spread over 3 city blocks encompassing 20 acres and involves the demolition and redevelopment for mixed use retail, office and residential properties directly adjacent to Historic Temple Square, (Figure 1).

Prior to construction, Block 76 was occupied by an existing shopping mall, subsurface parking structure and office buildings. Demolition of these structures, including the implosion of the Key Bank office tower, made way for the new construction. However an adjacent light rail system and four existing buildings between 9 and 23 stories remained at the corners of the project (Figure 2).

Support of excavation (SOE) work generally requires significant overlap of activities with the excavation contractor and therefore definition of work scope and schedule. A significant level of complexity was superimposed for Block 76 because of the extensive demolition required. The fact that new, deeper basement walls would have to be constructed along the alignment of existing basement walls required coordinated activities among all three subcontractors. For this reason, Nicholson met with Grant Mackey Demolition and Reynolds Brothers Excavation before the bid and spent significant time to adequately define scope and to coordinate schedules. The goal was to optimize the construction processes while accounting for costs. This effort may seem obvious, but Construction Managers often

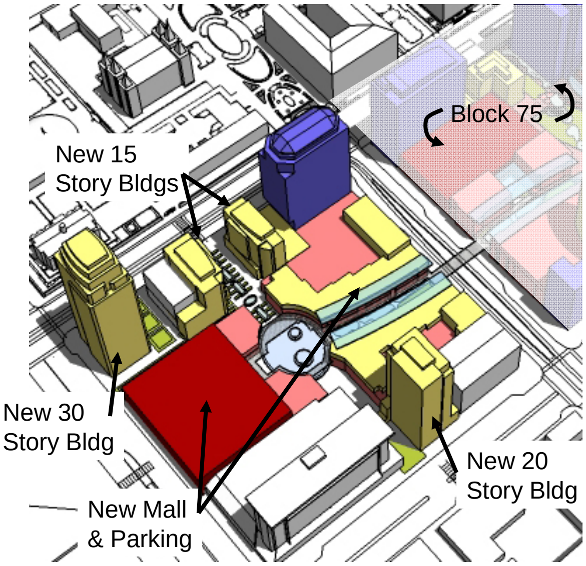

Figure 1 – New Construction at Block 76 of the City Creek Center Project

require individual price proposals from each subcontractor for the purpose of mixing-and-matching the subcontractors believing it will lead to the lowest overall cost to the owner. Fragmenting teams on a complex and technically challenging project is generally short sighted because the new grouping of subcontractors have not coordinated their activities and adequately defined each of their scopes and schedules. The cost savings are an illusion, because the projects generally have more scope and coordination conflicts, which lead to the submission of change orders and schedule delays. In the long run the Owner rarely gets best value with this approach.

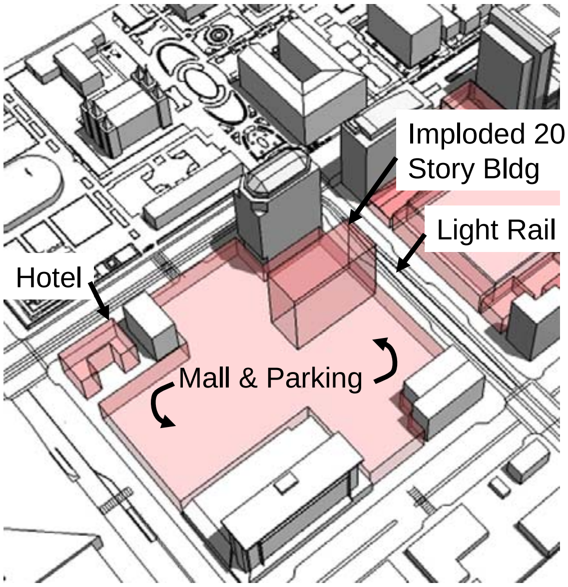

Figure 2 – Block 76 Demolition

Upfront challenges for the Block 76 project included:

1. Rigid geometric constraints for the SOE layout. Essentially, the inside face of the retaining walls were required to be 1 ft off the property line.
2. Maximum deformations were specified as 1 inch when excavation depths were up to 65 ft, and immediately adjacent to buildings up to 20 stories.
3. The reported clay foundation strength was borderline from a base stability standpoint.
4. The geotechnical profile was based primarily on historic data and perimeter investigation because there was very limited access for new borings/data within the urban footprint of existing buildings.
5. Groundwater was within the excavation depth on the northeast portion of the site.

SOIL CONDITIONS

The soil stratigraphy consists of layered alluvial and lacustrine deposits. The uppermost layer was medium to very dense sand and gravel laid down as alluvial fans from the adjacent Wastach Mountains. This gravel layer was underlain by medium to stiff clays and silts under which is a very dense layer of sand and gravel. The clay and lower gravel layer are deepest at the southwest corner of the site and rise towards the northeast corner of the site. The depth to existing groundwater is approximately 40-54 ft below grade with the highest groundwater at the northeast corner of the site. Figure 3 depicts the subsurface profile. (Image from Geotech report prepared by AGEC).

DESIGN CONSIDERATIONS

Excavation was required up to 65 ft below street grade and by as much as 50ft directly below adjacent buildings supported by shallow foundations. Controlling movement of the earth retention system and adjacent buildings was of critical importance to the project. The deformation constraints initially specified were the same across the site, even though the tolerance to movement varied substantially with adjacent property usage. An initial objective of the owner was to have the inside face of the support of excavation to be positioned within 1 ft of the property line at all locations. This proved unrealistic in some locations and very challenging at others. The challenges included the presence of existing basement walls that had to be removed while constructing the support of excavation at the same position to substantially greater depths.

Several meetings were held with the Owner and the CM in an attempt to balance cost and risk. Issues included better definition of the foundation clay strength, the ability to stay within the 1 ft geometric constraints for the

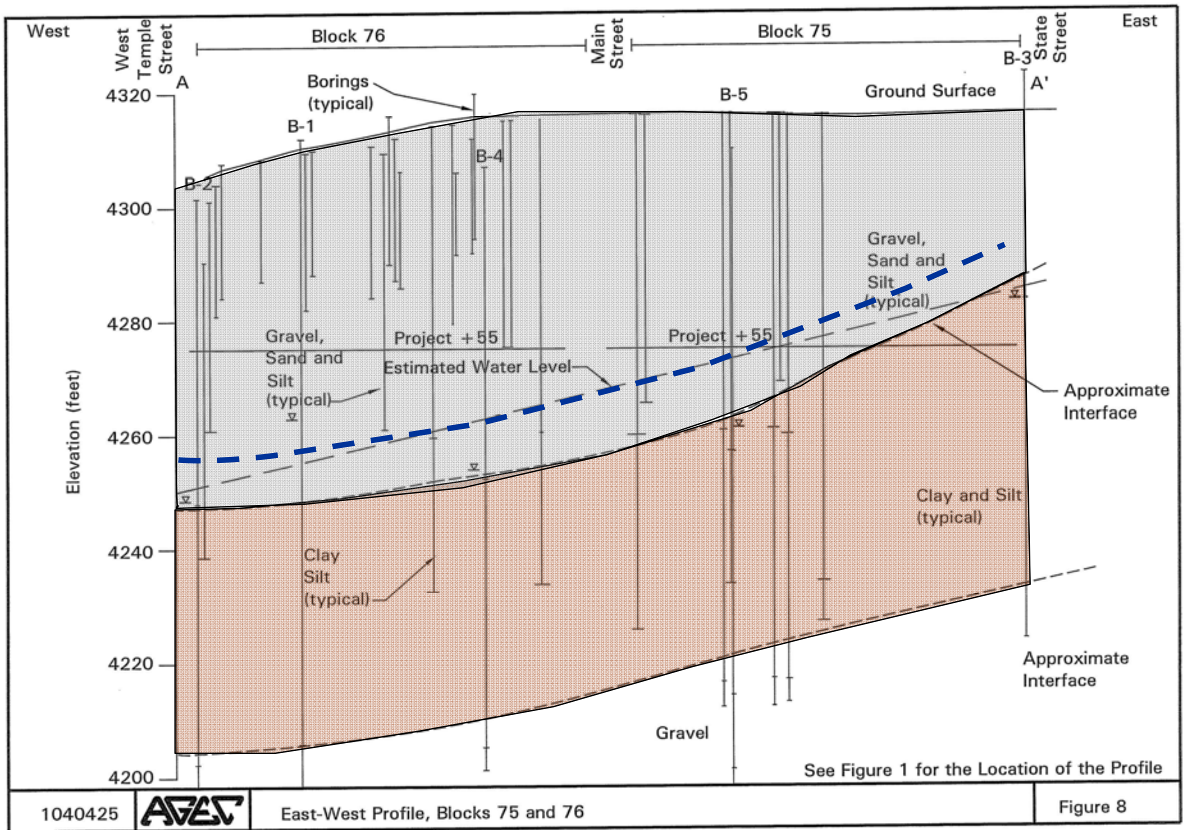

Figure 3 –Idealized Subsurface Profile.

SOE wall adjacent to tall buildings, and the added cost of mandating 1 inch deformation at all locations. An additional point of discussion was dealing with contingency costs.

The first challenge was to determine the strength of the clay stratum to preclude the risk of base heave. The shear strengths determined from available unconfined compressive strength data indicated that heave was a significant risk. However, it was believed that sample disturbance led to an underestimation of shear strength. Vane shear tests were undertaken by the Owners geotechnical consultant at a location adjacent to the site to better characterize the clay foundation. These vane shear data showed that an adequate safety factor against base heave was present in the clay base.

Through team meetings the owner accepted that a wall thickness of greater than 1 ft would be needed adjacent to tall structures in order to effectively manage risk. It was also acknowledged that the added cost of assuring 1 inch of deformation was not needed along the entire earth retention alignment so that more economical solutions could be developed.

The other point of negotiation was contingency risks. Contractors commonly include contingency costs to cover risks that cannot be accurately quantified. Perceived risks for Block 76 included:

1. Limited geotechnical data to adequately evaluate wall movement adjacent to critical buildings (because the site was covered with existing buildings);
2. Cut face stability of the granular soils during SOE construction;
3. Effectiveness of the dewatering program.

As an example, the SOE contractor can "lay" the responsibility of the dewatering effort on the CM or GC, but this does not eliminate risk. If difficulty in dewatering leads to poor bench conditions or project delays, it is highly unlikely that the SOE contractor will recoup real costs, and this process of negotiation can strain otherwise healthy relationships.

For Block 76 the Owner took a relatively unique approach in that it accepted these risks and chose to carry the contingency in its budget. Thereby, if the risks did not materialize the Owner would gain the benefit of lower cost.

SOE APPROACH

Anchored diaphragm walls were employed in the areas immediately adjacent to the existing buildings 9 stories or greater in order to meet the deformation criterion of 1 inch. The diaphragm wall design incorporated 3 to 4 levels of 6 strand tie-back anchors to control movement. A compromise was reached with the Owner in regards to diaphragm wall thickness and encroachment on valuable retail space which led to the design of a 24" thick reinforced concrete wall versus the 30" wall originally envisioned.

Soil nail wall systems were utilized at areas of the site not supported by diaphragm walls. The locations of soil nail walls were generally adjacent to streets where larger wall movements (up to 2 inches) were negotiated with the project team. Because the soil nail walls were not permanent, self drilling hollow bars were utilized in the design. Shotcrete facing was employed with wire mesh reinforcement.

Soil nail walls were also employed directly below existing structures of 3 stories or less, but were enhanced by an underpinning system consisting of micropiles with an integral cap beam to meet the specified limit of 1 inch of settlement. The underpinning system was designed to transfer the building loads to an elevation below the earth retention system.

The idealized subsurface profile in Figure 3 shows that the excavation to project Elevation 36 ft would be below the existing ground water at the north east corner of the site. Jet grouting was employed as an alternate to dewatering to mitigate water inflow along the north and east sides of project (Figure 6). A series of overlapping columns was designed to create a cutoff wall in between diaphragm wall segments along the north east part of the site.

The entire earth retention system and all existing adjacent structures were monitored with Sol Data's Cyclops real time automated monitoring system (Lange 2008). This system was a design requisite and an overall risk management tool for the project team.

CONSTRUCTION

Soil Nail Walls: The project began with a demolition phase that was supported by soil nail wall construction. As subsurface demolition occurred, the soil nail walls were sequentially installed as earth retention to support further demolition below grade. Figures 4 and 5 show the concurrent demolition, soil nailing and micropile operations

Figure 4 –Concurrent Demolition and Soil Nail Wall Installation**.**

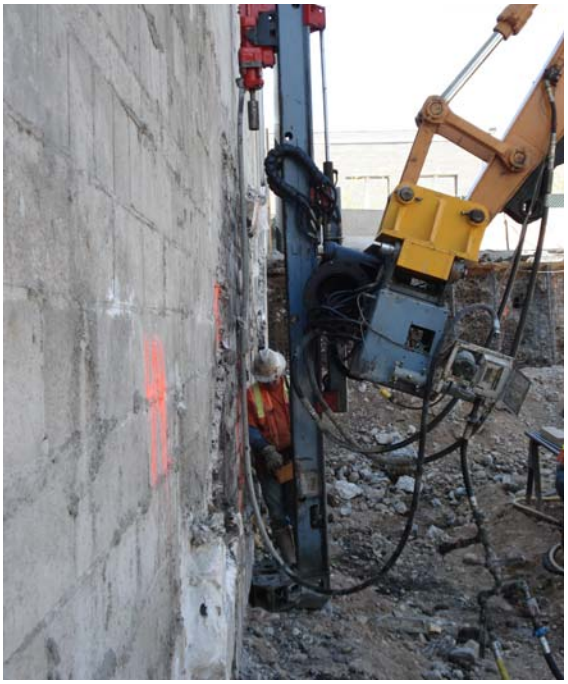

Figure 5. Marriott Hotel Underpinning

The soil nail walls were constructed using an excavator mounted, top drive rotary percussion drill. The excavator mount provided flexibility with drill setup and tracking over variable terrain. In addition, the drill mast has 360 degrees of movement making setup and installation at corner locations easier. As demolition areas were completed, a more productive pace was realized and soil nail wall construction was largely governed by the excavation progress. Over 5,000 soil nails were installed totaling over 150,000 lf of drilling and over 104,000 sq. ft. of shotcrete facing.

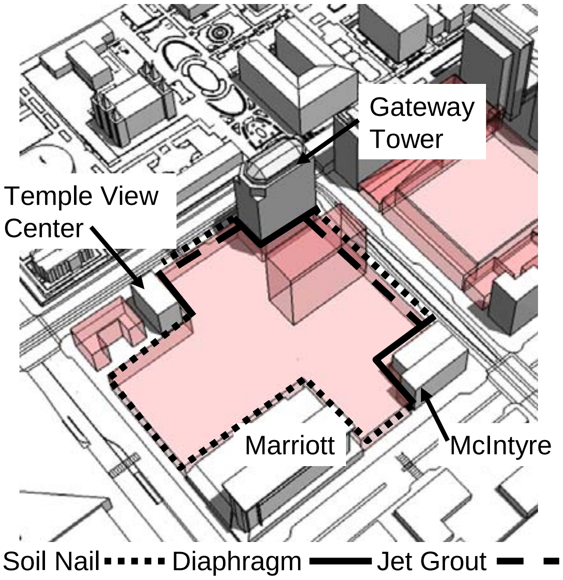

Figure 6 –Earth Support Techniques

During the initial soil nail sequence, underpinning activities were underway along the Marriott Hotel. (Figure 6 shows the earth support technique key plan). The use of hollow bar micropiles at this location allowed the piles to be installed closer to the existing wall because of a smaller drill head. Only a one foot clearance was available from the existing building to the backside of the new construction. To mitigate potential ground loss under adjacent footings, the drill rig also installed inclined grouted bars to aid the first 3 lifts of soil nailing. Figure 5 shows the actual installation for the Marriott Hotel underpinning. Micropiles were installed to a depth of 102 ft below grade and were bonded into the lower dense gravel formation. A total of 103 micropiles and 850 linear feet of cap beams were installed on the project at 3 locations; Marriott Hotel, Temple View Center, and at the Main Street Garage.

<u>Jet Grouting:</u> As discussed above, jet grouting was selected as an alternate to dewatering along South Temple and Main Streets. Two jet grout walls were constructed with a single row of overlapping secant columns and connected the diaphragm wall segments between the Temple View Center Building and the Gateway Tower on South Temple Street and between the Gateway Tower and the McIntyre Building on Main Street. By connecting to the diaphragm walls, the northeast quadrant of the site was essentially cut off from major water inflows.

The jet grout columns were installed so the bottom of column was keyed into clay about 5-10 ft below the base of the excavation and the top of the column was installed to 5 ft above the known high water elevation determined from the observation well readings. The average drill length for each column was approximately 50 ft and the jet length was 30 ft. A total of 236 columns were installed in a 2 month time frame to complete the water control activities enabling completion of soil nail walls below the ground water table. Figure 7 shows the exposed jet grout wall and completed soil nails.

Figure 7 –Jet Grout Wall South Temple Street

<u>Diaphragm Wall:</u> Diaphragm walls were constructed adjacent to the Temple View Center Building (9 stories), the Gateway Tower (23 Stories) and the McIntyre and Crandall Buildings (9 Stories). Each building is on shallow foundations, and excavation was required to approximately 40 ft below the foundations.

Figure 8 –First Anchored Diaphragm Wall in Salt Lake City June 2007

MONITORING OF EARTH RETENTION SYSTEM DURING EXCAVATION

The overall scope for the SOE included the monitoring, collection, and reporting of geotechnical instrumentation on the project. The Sol Data Cyclops system provided the real time monitoring of the shoring systems and building movements. Traditional inclinometers, observation wells and surface settlement points were reported on a weekly basis. Evaluation of earth retention system and building movements were done routinely by our Engineer, GEI Consultants, and Nicholson's project and home office staff.

Movement of the soil nail walls was expected to follow the typical pattern of rotation about the toe, similar to a gravity or cantilevered retaining wall. Maximum movements were expected at the top of the walls, with magnitudes in the range of 1 to 1.5 inches for the deepest cuts of 65 feet. Observed movements were typically much less than expected, generally by more than 50%. We attribute the good performance of the soil nail walls to the presence of the upper gravel stratum and its slightly cemented nature which was conservatively neglected in the design. In isolated areas where the upper gravel stratum was disturbed through placement of backfill for adjacent structures, the observed deformations were closer in magnitude to the anticipated values.

Movement of the diaphragm walls was expected to follow a pattern of increasing deflection with depth, corresponding to greater driving loads and very stiff ground in the upper stratum. Maximum movements were expected between mid-height and the toe of the walls, with magnitudes just under 1 inch for the deepest cuts of 40 feet. Observed movements were typically in line with expectations, although additional movement control measures were required in one portion of the site, which is described in the next section.

CHALLENGES AND SOLUTIONS

Face stability was a challenge for construction of the soil nail walls in some areas of the project. The material was an excellent representation of surficial geology associated with an alluvial fan deposits (see Figure 9). The coarsest openwork materials would ravel when cutting along directions adverse to the bedding orientation (on north walls). The solution was to fill voids with flow fill after wall placement. The limitations of Standard Penetration Tests are apparent in this environment. Locally, the Modified California sampler is also used, which is an improvement, but still has significant limitations in identifying the extent of the coarsest material in this cobbly ground..

Groundwater inflow proved to be a challenge at the connection between the diaphragm wall for the Gateway Tower and the jet grout cut off below the Main Street Garage. An electric duct/vault was present at this location requiring that the jetting be installed from compound angles to try and achieve a seal. Upon excavation it was found that a complete seal was not obtained and some secondary grouting was required using cement and polyurethane grouts. Figure 10 illustrates the stone fill placed at the leakage point to control "lost ground."

The most significant challenge came with the diaphragm wall for the Gateway Tower where unexpected movements occurred during the cut from the third to fourth level of anchors. Notification of this movement was obtained from the automated monitoring system and confirmed by immediate reading of the inclinometers.

Along the Gateway Tower, the anticipated soil conditions were the upper gravel stratum to the bottom of excavation with clay below the excavation depth and beyond the toe of the diaphragm wall. This expected profile was

Figure 9. Open coarse cobbly zones (top) and lost ground zones (bottom).

Figure 10. Seepage inflow at the connection between the diaphragm wall and the jet grout cutoff.

consistent with the observations during construction at other locations on the site and consistent with the characterization in the geotech report (Figure 3). However, the local conditions were quite different.

The first indication of differing soil condition came during tieback installation on the 3rd row when post grouting and supplemental anchors were required to achieve design loads. Upon completion of these anchors, excavation to the 4th level of tiebacks took place. Following excavation, the Sol Data Cyclops system recorded a ¼" movement at the diaphragm wall, which was corroborated with an inclinometer reading. This deflection was more than anticipated in this short time frame. A soil berm was immediately placed in front of the diaphragm wall to arrest the movement and provide time to understand the anomaly.

Additional soil borings and subsequent lab testing was performed to identify actual soil conditions. The additional subsurface investigation identified a 20 ft zone of inter-bedded silt and clay with numerous sand seams between the upper gravel and clay. This inter-bedded soil zone impacted stability of the wall in three ways:

1. The ground behind the wall was not as strong or as stiff as anticipated, requiring the anchors to carry more load;
2. The anchor bond zone lengths (and loads) were not appropriate for these materials and;
3. The soil within the passive toe was effectively drained, in contrast to the undrained response in the toe that was anticipated, and that was present at other locations on site. For comparison the difference in passive resistance of the toe between an undrained clay and drained silt was approximately 80 kips per lineal foot of wall.

Further complicating the analysis and field implementation of supplemental anchoring is the fact that the diaphragm wall alignment included a re-entrant corner and intersecting anchors.

The west wall proved to be on the critical path, but also provided the opportunity for an alternate solution to additional anchors. A large concrete mat slab was being constructed for Towers 6 & 7 at a distance of 40 ft from the west wall. It was decided to use subsurface struts to tie the toe of the diaphragm wall to the mat. These were installed sequentially in narrow excavated trenches to retain the passive load of the earth fill.

A construction sequence was developed where one strut was excavated, rebar dowels were drilled and grouted, a prefabricated reinforcing cage was hoisted in, concrete was placed, and the entire area backfilled during a single shift (see Figure 11). This process was used to construct 7 struts, and upon sufficient cure time, the soil berm was removed and the new construction was completed.

Figure 11. Sequential installation of subsurface struts at west wall of Gateway Tower.

Along the south wall a soil nail approach was used by installing long hollow bar nails on tight spacing in order to stabilize the wall. This approach had several advantages compared to simply adding more strand anchors:

1. The excavation could be made sequentially with shorter lifts minimizing loss of passive resistance.
2. The installation process and bonding of the nails was faster than conventional anchors because the hollow bars are installed quickly and the anchors would have required post grouting to achieve design loads. This allowed load to be carried faster, which was important because the passive resistance decreased with drainage of the silts.
3. The amount of load carried by each nail was less than that of an anchor and thereby more reliably achieved and providing more redundancy.

The approach employed performed very well with only small additional movements occurring during and after nail installation. Figure 12 depicts a wide angle view of the project and Gateway Tower.

SUMMARY

Block 76 is one portion of the large City Creek Center development underway in Salt Lake City. The design-build earth retention scope included diaphragm walls to control movement adjacent to large structures and soil nail walls at other locations. Micropiles were used for underpinning some structures and jet grout was used to reduce groundwater inflow.

The design-build approach evolved over a series of meetings with the Owner; the Construction Manager; and the excavation, demolition and SOE subcontractors. These meetings assured that scope and schedule requirements were adequately defined. The Owner was informed of areas of perceived risk and where contingency allowances should be budgeted into the program cost. In the end, the Owner accepted the financial risk for the contingencies identified. The adjustments required during construction as described here were paid from this contingency allowance through negotiation with the Owner.

The automated monitoring system was very effective for the project. Initial notification of movements along the Gateway Tower allowed for rapid response and controlled remediation.

Overall, the project successfully achieved the specified deformation and geometric performance requirements, and met schedule. Cost savings were realized by relaxing the initially restrictive deformation performance requirements where not needed, and by acknowledging and making allowance for contingency risk.

REFERENCES

LANGE, C and KIPPELEN, D. "Real-Time Survey Monitoring for Support of Excavation" Geo-Strata, Jan/Feb 2008.

Figure 12. General site view looking north.