DESIGN, PREDICTION AND PERFORMANCE OF TIED BACK SHOTCRETE WALLS SUPPORTING MASONRY HOSPITAL STRUCTURES

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ABSTRACT

Two recent projects were completed in Ontario, Canada for major hospital additions that involved excavation support of heritage masonry structures. Conventional continuous caisson wall schemes were replaced, with significant cost savings, by tied back shotcrete systems. Using a combination of conventional design and FLAC analysis, excavation support was designed to permit excavation in soft and competent soils that limited horizontal movements of the existing structures to between 3 and 8 mm. The use of vertical wall elements (micropiles) integral to the shotcrete wall to support both the vertical wall loads and the face of the excavation was deemed to significantly reduce lateral and vertical wall movements. FLAC analysis was used successfully to predict wall movements and optimize design. The shotcrete method proved to be flexible and adaptable to design changes during the project. This permitted convenient accommodation of changes to the building geometry after excavation support had commenced. In addition the method accommodated staged demolition and excavation support of portions of the existing structure at low cost, conveniently and without delays.

INTRODUCTION

Design build schemes using tied back shotcrete shoring were recently used to replace conventional caisson wall excavation support at two prominent structures in Southwestern Ontario. Shotcrete shoring offered cost savings, arising mainly from lower material costs, and economies related to the use of light construction equipment deployed with relative ease. Tied-back shotcrete walls were approximately 20 percent of the thickness of conventional shoring walls. Smaller equipment, lower concrete volumes, fewer compressors, and less truck traffic contributed to lower dust and pollution levels, a significant hazard in a hospital environment, particularly in facilities treating transplant patients or patients with respiratory difficulties. Compact and versatile, the installation equipment was ideal for working in close proximity to adjacent buildings, on steep grades and through intermittent or staged construction requirements.

The sensitivity of the neighbouring structures led the design team to set stringent lateral movement limits. Performance of the excavation support required enhancement over conventional tied back shotcrete in order to achieve the desired deflection limits. A high cost solution was to add anchor length and decrease panel sizes. Alternatively the design team sought new methods of improving system performance.

The design team used FLAC analysis to optimize design and offer insight into the performance of the shoring and neighbouring structures. FLAC analysis was used to model several different geometries and included varying the anchor bond and
free lengths, inclusion and exclusion of micropiles behind the face of the wall and varying the spacing and depth of the micropiles. The FLAC method was successful in identifying possible performance enhancement, but required field verification.

Extensive use of instrumentation was made to assess the performance of the shoring during and following installation. The information provided enabled optimization of the design during installation and alerted the designers to problems before they became critical. The lack of precedent and monitoring data for tied back shotcrete shoring systems in weak soils was a concern at the outset of the projects. The design build team was responsible for system performance and considered proper treatment of the variable soils and vibration and deflection sensitive neighbouring structures a challenge at both sites. Instrumentation included inclinometers, conventional and precision survey monitoring, load cells mounted beneath anchor heads and electrolevels positioned in horizontal and vertical arrays perpendicular and parallel to the shoring. Shoring modifications were relatively easy to effect during construction due to the inherent flexibility of the shotcrete method.

The performance of the walls was found to be good to exceptional, in some cases exceeding the anticipated performance of much stiffer structural systems. Cost savings and adaptability to changes in the geometry of the existing and proposed structures were significant advantages of the shoring system used. In addition advance placement of 1st footings was achieved and challenges with staged demolition and site access were overcome.

**DESIGN**

Both conventional design and advanced analytical methods were used to evaluate the shoring design at each structure. Conventional design established initial shoring loads and anchor geometries. Advanced design methods were used for optimization and enhanced analysis in problem areas. This included prediction of movements in areas where the neighbouring structures were very close to the proposed excavation and where unusual soils conditions existed whose behaviour was difficult to anticipate.

Conventional tied back shotcrete shoring design methods offer little prediction of shoring performance and ground movement. Nor do they accommodate variable soils conditions in a comprehensive manner. Issues of movement prediction and localized areas of weak soils are left largely to the engineer’s judgment. Prediction of performance and an understanding of the effects of localized weakness in the soils were of paramount importance. Sensitive eye surgery equipment existed within the neighbouring structure at the Brantford site and sensitive laboratory testing was performed adjacent to the MaRS Centre B site.

**Brantford General Hospital, Brantford, Ontario**

Recent additions to the Brantford General Hospital (BGH) in Brantford, Ontario included construction of a new hospital wing, involving excavations of up to 11 metres (36 feet) depth, in loose to compact sand adjacent to an existing eight-storey hospital structure. The critical section is shown in figure 1.
At BGH the geotechnical report indicated the sand was fine grained, grading to fine to medium, with very low silt fraction and moisture content of 1 to 9 percent. Grain size distribution curves for the soil are shown in Fig. 2. Standard penetration test (SPT) and dynamic cone penetration test (DCPT) results indicated the sand was loose to compact near the surface, becoming increasingly compact with depth. The fine grained sands encountered were poorly graded with little silt fraction and as a result when excavated the shoring face was stable for a very short period of time. This was anticipated by the design team and design measures were taken to assist in face stabilisation during construction. The control of the face was recognised to contribute significantly to the overall shoring wall performance.

For protection of the adjacent buildings, the design-build team set a 6 millimetre target limit on shoring deflections. FLAC analysis indicated this level of performance could not be achieved without additional stiffness being introduced into the shotcrete system. Thicker shotcrete did not appear to offer reduced movements because it still involved leaving the shoring face exposed. Smaller panels improved the performance but were deemed not cost effective. Analysis indicated the inclusion of vertical minipiles behind the shotcrete wall would both enhance the stiffness and carry any vertical loads down to the base of the excavation. The inclusion of minipiles was thought to offer the added
benefit of excavation face control and was adopted. Thus the final design included additional soil face protection measures, a detailed tieback stressing program and monitoring to permit real time adjustment of procedures to reduce movements.

The program included:

- Vertical dowels, consisting of steel bars in 3 inch drilled holes, installed at the shoring line prior to the start of excavation. The dowels, approximately 3 per panel, provided temporary face support during excavation and shotcrete application.
- Dowels in 8 inch holes on 4 foot centres augmented the smaller dowels at the most critical part of the excavation, to provide vertical support for the shoring wall should ground loss occur.
- To minimize ground loss potential, self-drilling hollow bars were installed and grouted to surface with sleeves to obtain design free zone lengths. Tiebacks were partially stressed the morning after panel construction, and fully stressed prior to excavation of the next lift.

Fig 2. BGH Fine Sand Grain Size Distribution Chart

The excavation was completed on schedule, with cost savings of 20 percent over a conventional caisson wall and soldier pile and lagging shoring solution.

Particular attention was paid to stressing the tie backs, mapping load distribution along the tie back length and changes in tie back load distribution with project duration. All tiebacks were
proof-tested by cyclic loading to check free length and anchor performance. Early in the project, it was noted that elongation values in a significant number of proof tests indicated less-than-design free lengths. Where this occurred, the tests were repeated using higher proof loads to break bonds and mobilize longer free zones. Lift-off tests were performed wherever loss-of-load was suspected, and to ensure inspection records were complete. In total, 4 percent of tiebacks were lift-off tested. Seven random lift-off tests, conducted on upper level tiebacks when excavation depths were 20 to 25 feet, indicated tieback loads were 86 percent of design load on average. Load cell data was obtained at the third and fourth tieback rows which indicated tieback lock-in values were 110 and 114 percent of design load respectively, and initial load losses were 18 and 22 percent of lock-in values respectively. With both load cells in place, the remaining 20 feet of soil was excavated in 45 days and additional load losses of approximately 5 percent of lock-in value were measured. Final load cell readings, taken one month after excavation was completed, indicated tieback loads were approximately 84 percent of design load. At the maximum test load of 60 kips, anchor forces ranged from 1.1 to 2.6 kips/feet. The fourth test, carried out on a non-production tieback with the bond length shortened for testing purposes, demonstrated an anchor adhesion capacity of 7.3 kips/feet.

The deepest part of the excavation coincided with the highest part of the BGH hospital structure and was instrumented with precision survey targets, an inclinometer (NE), an electrolevel (W8), and two load cells. Inclinometer plot NE, displayed in Fig. 3, showed characteristic into-site movement concurrent with excavation of each lift. The time period between lifts, from initial berm excavation to final tieback stressing varied from four to fourteen days. The plot indicates maximum displacements were localized in areas of active excavation near the base of the cut. Above the active lift, where shotcrete application and tieback stressing had been completed, displacements were negligible; note less than 1 millimetre of movement occurred at the top of the wall between October 4 and December 12.

Upon completion of all excavation and shotcrete wall construction, the maximum relative into-site displacement was 2.2 millimetres. Additional post-excavation displacements, mainly
attributed to compaction vibrations, increased the maximum displacement to 3.3 millimetres or 0.03 percent of the excavation depth. Of note, the adjacent hospital structures' movements were measured as less than 3 millimetres, better than expected from a caisson wall system. The excellent performance of the shotcrete shoring in the hospital wing phase was attributed to shoring design features, good workmanship, and rigorous quality control efforts by the design-build team.

**MaRS Centre B, Toronto General Hospital**

The success at BGH prompted the design team to undertake another design build job at the MaRS Centre B expansion of the Toronto General Hospital. The job involved an infill structure between two heritage structures and two modern structures, one a laboratory and the other a hospital wing. The critical section of the site included excavation an depth of 28 ft (8.5m) with footing line loads of 19.2 kips/lin ft (280 kN/lin m) either underpinned or within 3 ft (0.9 m) of the excavation face. An in depth FLAC analysis was conducted to determine the need for and/or optimum spacing and length of the shoring face minipiles. The FLAC analysis indicated the minipiles could be spread out and shortened from the geometry used at BGH.

The soils at the site consisted of silty clays and competent clayey tills, which increased the courage of the design team. However CPT results obtained post tender to gather improved soil date, indicated the presence of a weak soil layer at a depth of 2 m to 3.5 m below the existing structure footings. The result became a wash, with the weak soils taking away what the analysis provided in improvements. To complicate matters the first two levels of anchors indicated poor adhesion with capacity ranging from 90 to 120% of design values A typical section is shown in figure 4. Varying of the minipile depths and spacings was planned to investigate their effect on shoring wall performance. However, the poor and variable performance of the anchors created difficulty in establishing the effects of changes in the minipile geometry.

The CPT results shown in figure 4 compared with the conventional soils investigation indicate the disparity in the results from the two sampling methods. The CPT picked up the weak soil layer quite clearly whereas the SPT counts indicated a very dense layer. Eventually drilling techniques were modified resulting in improved anchor capacities in the upper two rows. Towards the end of the job the design team was better able to explore field experimentation and optimisation of the minipile configuration. The results were very informative.
Figure 4 Typical section at MaRS Centre B Toronto General Hospital, and CPT results.

Figure 5 indicates the response of the shoring wall where the minipiles were installed in a pattern with varying lengths. The short–short-long pattern consisted of minipiles at 0.75 m centres with every 3rd minipile extending 1.5 m below the base of the excavation. The intermediate short piles terminated one lift, 5 ft (1.5 m) above the final excavation level. The shoring wall performed well as the excavation progressed with expected increases in wall displacement creating the characteristic ‘bulging’ with each lift. The final lift resulted in more pronounced overall wall displacement and peak movements of 12 mm.

Figure 6 indicates the response of the shoring wall where every minipile, on a 0.75 m spacing, terminated 1.5 m below the final level of excavation. The shoring wall performed well as the excavation progressed, creating the characteristic ‘bulging’ with each lift right through to the final lift. Peak lateral movement was 9 mm. With time the movement increased to a total of 10 mm.
Figure 5  Inclinometer results for intermediate minipiles 1.5 m above excavation base.

Figure 6.  Inclinometer results for minipiles extending below the final excavation level.
FLAC analysis was used effectively to model the influence of minipile depth, but was less successful at modelling minipile spacing due to the 2-D nature of the analysis. In figure 7 the results of the FLAC analysis conducted on high and low minipile termination are provided. They indicate a total deflection of 8.8 mm for minipiles extending below the final excavation level and movements of 13.7 mm for minipiles terminating 1.5 m above the final excavation level. The FLAC results appeared to be influenced by the modelling of soil lateral response with the minipiles, number of calculation steps modelled between exposure of the excavation face and the placement of support and the length and stiffness of the minipiles. Evaluating the combination of these parameters proved to be demanding and, while successful for this situation, would require further calibration on future projects. In addition modeling of soil flow around the minipiles and an investigation of the effects of diameter, spacing and stiffness is recommended. Future attempts will be made by the design team to use FLAC 3-D modelling for this application.

Figure 8, FLAC results indicating shoring response to variance in minipile depth.

The flexibility of the shotcrete system proved valuable in accommodating a number of changed conditions on site as well as the staging of construction activities. At the MaRS B site an elevator shaft was to remain in service during the de commissioning of the neighbouring structures. During the construction schedule however it was taken out of service and demolished to accommodate the proposed construction. This resulted in the requirement to underpin the structure during the initial shoring schedule and then, during footing and column construction of the proposed structure, demolish the elevator shaft and remove that portion of the building. Thus a scheme was devised to place extra long anchors during the initial drilling which would serve as the future anchors following demolition of the elevator shaft. In this manner the demolition could take place, the initial shoring
removed and the soil supporting this portion of the structure could be removed using small equipment. As the excavation proceeded towards the future final shoring wall, the existing anchors were located and served as support for the new shoring wall. The scheme was simple, performed well and created little disruption to the ongoing construction activities.

Figure 9. Staged shoring and demolition of a hospital elevator shaft.

CONCLUSION

The substitution of tied back shotcrete shoring for conventional shoring was conducted at two large hospital sites in Ontario. The use of shotcrete decreased costs and increased the flexibility of the design team to accommodate unexpected site geometries, saving further costs and schedule delays. The use of FLAC 2-D modelling was instrumental in predicting the scope of movements of both the shoring face and the neighbouring structures. FLAC was also instrumental in evaluating improvements in the shotcrete design to enhance system performance while keeping costs in check. The use of minipiles behind the shotcrete face was instrumental in controlling shoring movements. Further optimisation of the minipile geometry is expected to occur on future sites with the use of FLAC 2-D and 3-D modelling to assist in the prediction of performance.

REFERENCES

PetoMacCallum Ltd, Consultant Engineers [1999]. “Geotechnical Report for Brantford General Hospital”.