The Bow, EnCana excavation support system design and performance

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ABSTRACT
EnCana Corporation's new office tower in downtown Calgary required an excavation 20.5 m deep spread over two city blocks. The excavation support system was designed to overcome the challenging geology of the area using the Observational Method, in-depth finite element modeling and prediction, and an extensive monitoring program. Results from the monitoring program confirmed modeling predictions and identified rock expansion mechanics and problem areas.

RÉSUMÉ
La nouvelle tour de bureau de Corporation EnCana à Calgary du centre a exigé une excavation de 20.5 m de profonde réparti deux blocs de ville. Le système de support de l'excavation a été conçu pour surmonter la géologie difficile par suivre la Méthode d'Observation, modélisation et prévision d'élément finie détailles, et un programme de surveillance intensif. Les résultats du programme de surveillance ont confirmé les prévisions de modélisation et ont identifiée les mécaniques de l'expansion du roche et les domaines problématiques.

1. INTRODUCTION

EnCana Corporation, based in Calgary, Alberta, is currently spread throughout five separate offices within downtown Calgary but will soon consolidate its employees and create a lasting impression upon the Calgary sky line with the addition of The Bow office tower. The Bow is located in the heart of downtown Calgary, with the Petro-Canada Centre to the west, the Telus office tower to the north, and Calgary's Light Rail Transit (LRT) line to the south. Spread over two city blocks, straddling 6^{th} Avenue, the complex is split into two sections, including a 54 storey Bow office tower on the north block and a cultural centre to the south.

The excavation support system was required to encompass an area of two city blocks with a depth up to 20.5 m. Challenges included the size, location, geology, and schedule of the project. The shoring system design and construction utilized the Observational Method through in-depth analysis and prediction with finite element modeling based upon available information on the surrounding bedrock, innovative shoring practices, and a detailed monitoring program during construction.

The site stratigraphy is composed of fill above 6 to 7 metres of well graded gravels and cobbles over bedrock composed of layered weak mudstones, siltstones, and sandstones, with the predominant mudstone highly susceptible to weathering. The lack of information regarding the in-situ stresses of the bedrock and the location and effects of shear band propagation in weak rock layers prompted extensive FLAC analysis. The design was supported by background research of case studies in the Calgary area combined with a peer review from local experts. Several geotechnical scenarios were predicted which accounted for varying residual rock stresses and the location and composition of weak rock layers suspected of causing shear band movement.

To measure the performance of the shoring wall a detailed monitoring program was implemented that included 12 inclinometers, 8 extensometers, and precision target monitoring of the shoring wall and surrounding buildings. During the course of excavation and construction, the monitoring results were used to determine the global and section specific performance of the shoring system, re-evaluate the bedrock properties for more accurate FLAC analysis, and evaluate the effects and magnitudes of shear band propagation.

The Bow project is currently the largest urban excavation in Canada, outside of Toronto, with a footprint of approximately $17{,}000 m^2$ and a shored face of $13{,}200 m^2$. Throughout the span of the project, over 4700 m^3 of concrete were used, approximately 6400 m^2 of shotcrete were placed, and nearly 1600 anchors were installed. 6^{th} Avenue, the division between the two sections, was demolished to allow for the construction of the six storeys of underground parking stretching across the entire site.

2. GEOTECHNIQUE

2.1. Site Conditions

AMEC Earth and Environmental performed geotechnical investigations at the site in 2005 and 2006. The ground surface at the 190 metre long site is generally level, with elevations ranging between 1045.5 to 1046.3 m. The site consists of 0 to 0.5 m of fill over 5.5 to 7 m of well graded gravel. The gravels and cobbles are well rounded fluvial deposits from glacial rivers. Bedrock was encountered between elevations 1039 to 1041 m, with localized areas reaching 1035 to 1036 m. Groundwater is located in the gravel soils and varies seasonally from 1 m to 3.5 m in depth above the rock surface.

The bedrock consists of the Porcupine Hills formation with layered mudstone, siltstone, and sandstone in nearly horizontal bedding. The siltstones and sandstones form indurated lenticular layers of varying thickness and extend throughout the majority mudstone. Formed as lacustrine deposits, the cementing agent of the mudstones and siltstones is clay, giving these formations a more soil like quality than the sandstone lenses. The mudstone varies from very weak to medium strength, with point load axial tests recording strengths up to 0.8 MPa. The siltstone and sandstone layers have medium to high strength, with point load axial tests recording between 0.16 and 5.14 MPa in the siltstone, and 1.7 up to 8.1 MPa in the sandstone. The mudstone is prone to rapid weathering upon excavation with exposure to oxygen and water.

2.2. Bedrock Properties

The mechanical properties of the bedrock formation are poorly understood. The Porcupine Hills formation is known to exhibit large lateral in-situ stresses ($K_0 = 2.0$) as well as susceptibility to shear band propagation in weak mudstone layers. The shear band phenomenon was suspected to have resulted in elevated lateral movements in the past at two nearby projects.

Table 1 provides the engineering properties used for the design and analysis. Effort was made to explore the range of parameters suggested by the local geotechnical review team. The parameters were checked against the conditions and range of movements observed at nearby Calgary deep excavations. Overall, the recommendations provided by the team were found to provide representative results when compared to the movements experienced at some Calgary deep excavation sites.

Table 1. Soil engineering properties assumed for the analysis.

Description	Bulk Modulus MPa	Poisson's Ratio	Internal Angle of Friction (Degrees)	C' kPa	Unit Weight kN/m^3	Dilat	Shear Modulus MPa
Soil cap	36	0.33	28	25	21	0	13.6
Gravel soils	73	0.33	36	0	20	2	27.8
Weathered Bedrock	104	0.33	26	50	22	0	36
Upper sound Bedrock	112	0.27	40	100	22	0	67
Lower sound bedrock	133	.285	40	125	23	0	75
	Normal k	Shear k	Phi				
* Shear Band*	10 MPa	10 MPa	12				

3. EXCAVATION SUPPORT GEOMETRY

The design was required to address issues typical to deep excavations in Calgary, such as soil loss in the gravels, excessive groundwater ingress into the excavation, weathering of the mudstone, and risk associated by rock mechanic driven movements.

The design incorporates a continuous caisson wall installed 2 m into the sound rock with an anchored shotcrete wall beneath in the excavated bedrock. This design eliminates typical soil loss in the gravels, aides in controlling groundwater ingress, prevents the weathering of the bedrock, and addresses the movements caused by rock strain. The design geometry is provided in Figures 1 through 3. The caisson wall consists of interlocking 880 mm diameter drilled, cased holes with W460 x 68 kg soldier piles set between 2 filler piles. All concrete had a minimum strength of 5 MPa. The filler piles extend 2 m into rock with a single filler per bay extending 4 m into sound dry rock, which created a smooth transition from the caisson wall to the shotcrete face and added vertical support to resist anchor loading. The toe of each pile is constructed of fortified caisson mix, where a minimum of 120 kg of concrete was added and mixed into the bottom 2 m (achieving an anticipated 15 MPa strength).

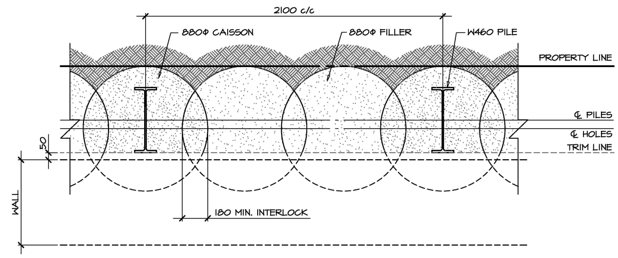

Figure 1. Typical caisson wall detail.

The shotcrete wall was constructed to act as a weatherproofing shield, guarding against deterioration of the mudstone bedrock. The excavation procedure for the shotcrete wall was controlled with a double benching procedure. The rate and staging of the berm removal was controlled by the shoring contractor such that all shotcrete was prepared and placed within 48 hours of exposure of the rock to the elements. The shoring contractor excavated the final 150 mm of rock using a concrete wheel abrator mounted on an excavator, producing a uniform, virgin rock condition prior to shotcrete application. Wick drains were placed against the rock face to drain rock borne water from behind the shotcrete face into the excavation, preventing deterioration of the mudstone bedrock.

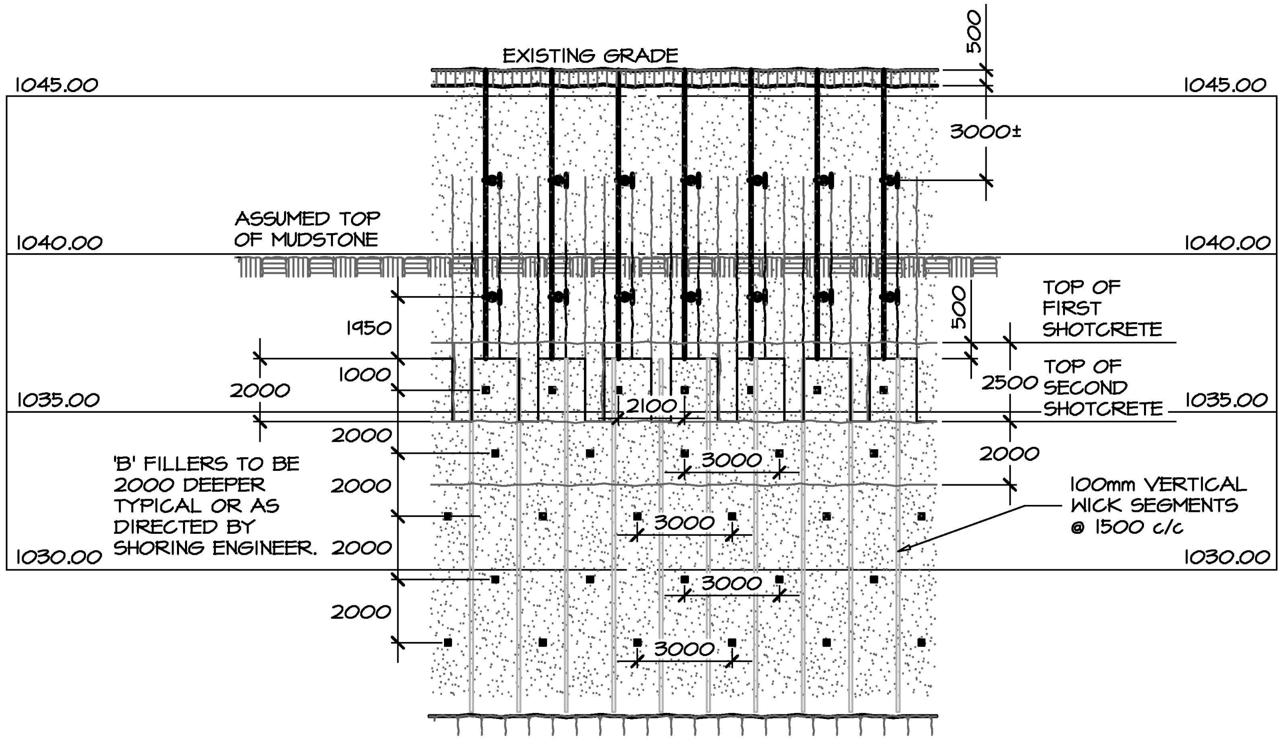

Figure 2. Typical elevation detail.

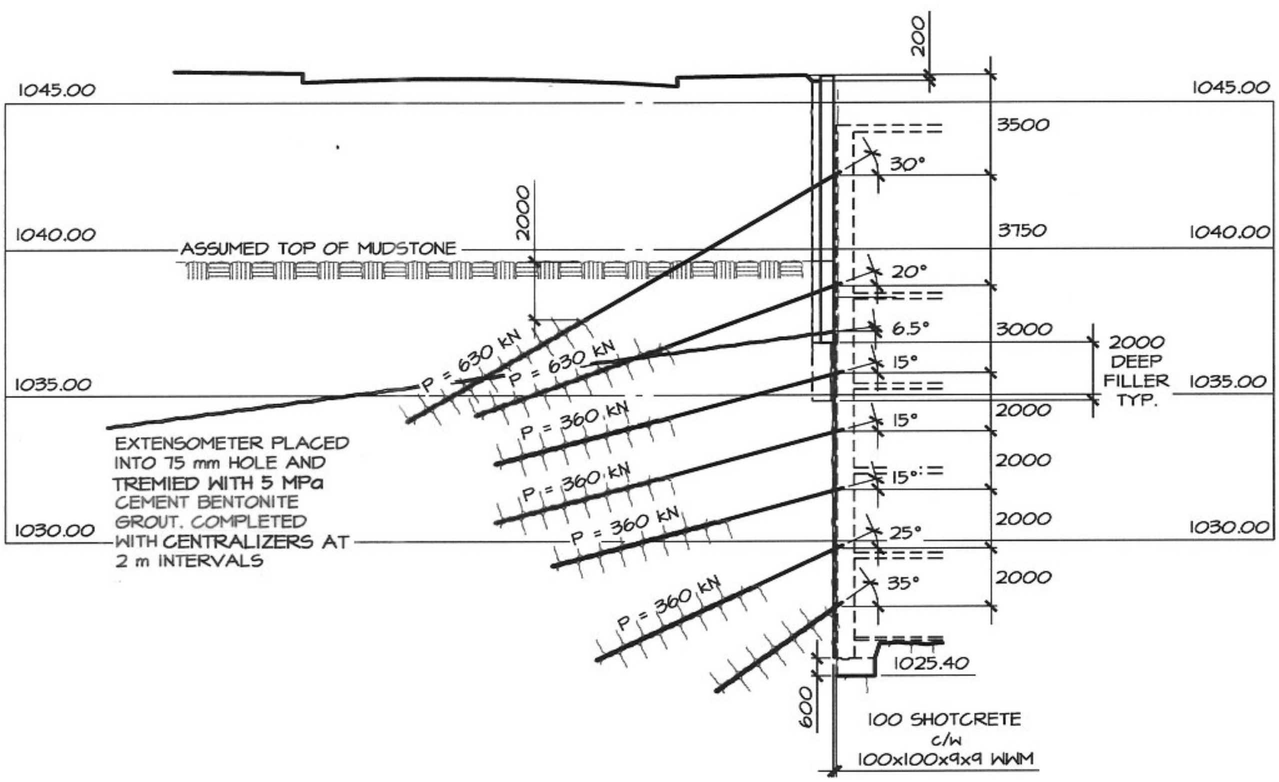

Figure 3. Typical cross section.

4. MODELING AND PREDICTIONS

4.1. Modeling of Bedrock and Shoring System

The proposed excavation support system was analysed using Two-dimensional Fast Lagrangian Analytical Continuum method (FLAC). The method has been used for geotechnical and mining engineering problems for over 15 years and has been well documented in literature. The 2D FLAC grid as modeled in the analysis is 105 m wide and 60 m in height with 13,700 discrete elements numbering 137 across and 100 high. The section modeled represents the typical geometry as provided in Isherwood Drawings with a depth of 20.5 metres. The grid is not symmetrical and depicts the final geometry to a distance of 50 m towards the centre of the excavation.

In order to better understand the rock properties an extensive analytical study was completed using case studies of excavation and shoring in the Calgary area and FLAC analysis. This was combined with a peer review session to help clarify the known processes and identify areas that required additional work. The case studies focused on excavations within the Porcupine Hills formation, and included jobs such as the James Short Parkade, City Parkade, and the Edmonton Convention Centre. The results from these projects were analyzed with the representative bulk rock properties being back calculated. Local literature and expertise supported two mechanisms for excavation support movement in Calgary: elevated in-situ lateral soil stress condition (assuming a K_o of 2.0) and 'shear band effect' where localised progressive failure of a weathered mudstone layer propagated movements. Parametric studies were conducted under a variety of assumed soil parameters, in-situ lateral stress conditions and weak layered rock conditions (shear band conditions at depth).

The anchor system was designed based upon the results from the extensive FLAC analyses. Given the lack of test data or confirmation regarding the strength, stiffness and mode of movement (internal lateral rock stress and/or shear band effects) of the Porcupine Hills formation a parametric analysis was undertaken to explore the proposed geometry and an assumed range of rock properties. The results indicated that movement was unlikely to be caused by either relaxation of high in-situ stresses or shear band propagation alone. This was confirmed by the presence of numerous excavations developed deep within the Porcupine Hills formation that did not witness excessive movements (movements greater than 30 to 40 mm). However, proximal sites had witnessed excessive movements (>75 mm) and thus the risk was present at the Bow, both due to its location and the sheer size, with the excavation along Centre Street up to 190 m in length upon completion.

The FLAC analysis established that in order to model movements of 75 mm or greater a combination of internal lateral stress and shear band effects would have to occur simultaneously. Given the inconsistency of the movements witnessed in Calgary deep excavations it was postulated that the effects combined infrequently. In order to counter these effects as efficiently as possible a strategy was developed using a flexible anchor program combined with an extensive monitoring system. The solution was designed to create a large block of stable rock, which would act as a gravity dam to resist strain in the rock. In addition the multiple rows of anchors would have a greater chance of engaging rock across a shear band. The top anchors were given large loads to prevent deformation in the stiff caisson wall supporting the soils. Many rows of smaller capacity anchors were used in the rock which acted to knit together the rock mass into a monolithic, stable block that would resist shear band effects.

4.2. Prediction of Shoring and Rock Behaviour

The predicted loads and movements for the proposed excavation support are highly dependent upon the assumptions made regarding the integrity of the bedrock (shear band effects) and the residual lateral load within the bedrock. A parametric study revealed predicted movements increase substantially with various combinations of weak rock layers and lateral rock stresses. The caisson wall structural system is affected little by the changes in the assumed rock behaviour. The shotcrete wall is expected to perform within the design envelope despite changes in the assumed rock behaviour. However, substantial changes in rock anchor loading and wall movement are witnessed with the programmed changes in the rock behaviour. The most notable outcome of the analysis was the resulting shift in emphasis from the in-situ lateral soil stress regime to the shear band effect as the major cause of lateral movement.

In keeping with the Observational Method, FLAC analysis was used to predict movements of the shoring wall and neighbouring structures. The progression of predicted wall movements with excavation is dependent upon the mechanism of the driving force / rock integrity. Lateral stresses in weak rock promote a progressive movement deformation profile. Shear band effects produce more sudden movements of the wall face as a whole. As the excavation level nears the rock surface, the shear band begins to reveal itself and manifest large movements within a plane of the rock mass. Thus the characteristics of the progression of movement as the excavation is developed influence the interpretation and reaction to increased rock movements. Figure 4 provides the FLAC analysis performed for a deep seated shear band mechanism at a depth of 5 m below the final excavation level. Note the movements at the shear band are on the order of 30 mm and the total peak face movement is predicted at 45 mm.

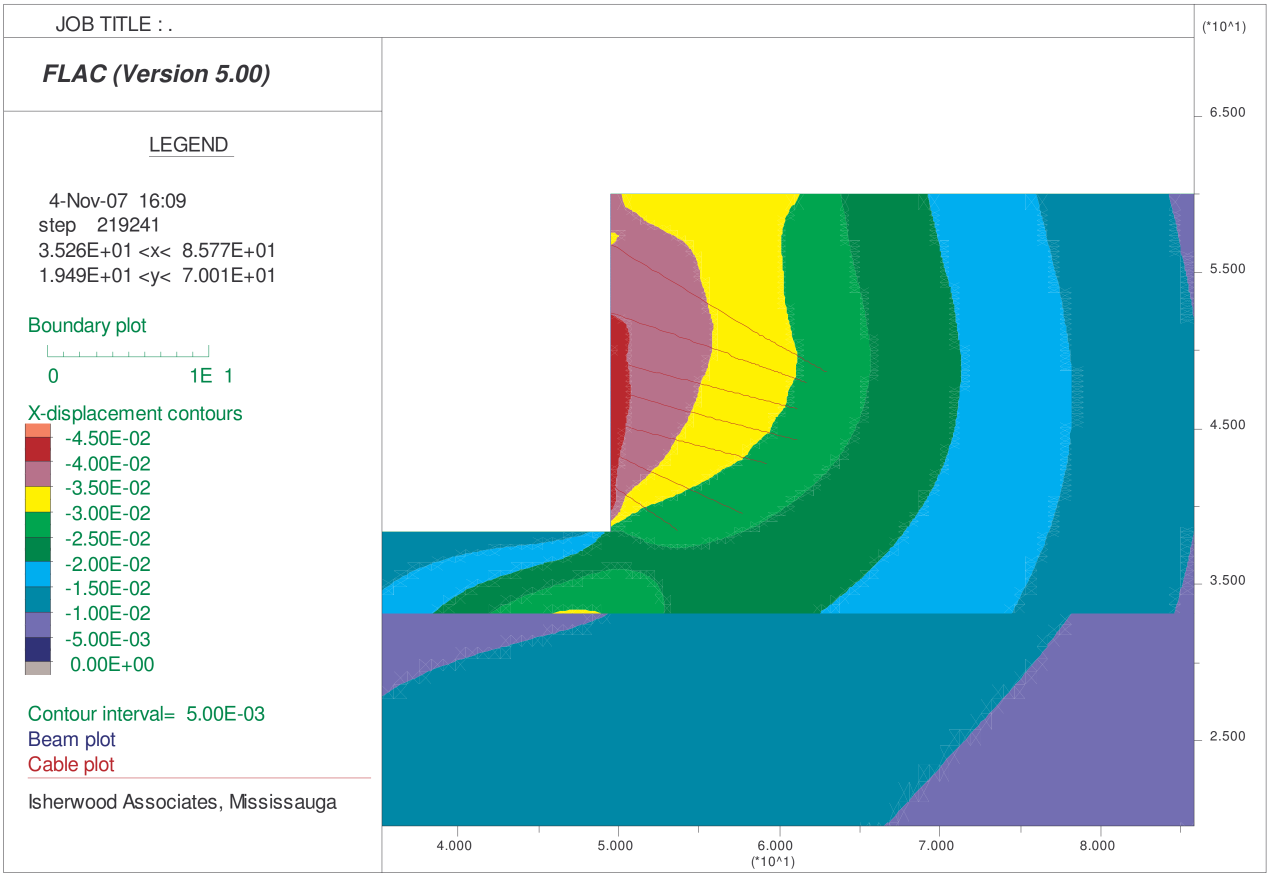

Figure 4. FLAC analysis for deep seated shear band showing horizontal displacement (contours at 5 mm)

The designers published a design prediction prior to the commencement of the project that provided projections of excavation face movements for three scenarios: baseline with K_0 = 0.6, shear band, and K_0 = 2.0. In keeping with the Observational Method, review and alert thresholds were also provided for each excavation stage. Based upon the characteristics and magnitude of the rock movements observed at the face, additional analysis or remedial action would be invoked to control the excavation performance.

5. MONITORING PROGRAM

The monitoring program implemented at The Bow project involved a combination of 12 inclinometers, 8 extensometers, and precision target monitoring of the shoring wall and neighbouring structures. The monitoring program for The Bow's excavation support system was designed to ensure the full and redundant observation of the shoring wall and rock movement. Monitoring instruments were clustered at locations chosen to accurately reflect the behaviour of the wall face at critical sections of the excavation.

Precision target survey was conducted at desired locations on the shoring system and surrounding structures. At the Bow, survey targets were placed at the top and bottom of each pile and at lower anchor rows along the shoring wall. The top and bottom of pile targets provided early indication of the caisson wall behaviour and confirmed movement readings of the other instruments. Targets on the shoring wall were reported weekly, while targets on the surrounding structures were reported monthly.

Inclinometers were read weekly to determine the horizontal movement profile in the vertical plane of the shoring face, both in the wall and well into the rock below. In general, the inclinometers were installed to a depth of 10 m below the final excavation to ensure any deep seated movements were captured.

In conjunction with the precision targets and inclinometers, extensometers were also installed at 8 locations across the site. The 25 m SMART cable extensometers are composed of 6 potentiometers that measure the relative movement of the rock mass with respect to the excavation face. The purpose of these instruments was to determine the distribution and magnitude of rock movement beyond the face of excavation. Placed just below the bedrock surface, the extensometers were installed at 7 degrees to horizontal and encased in a special grout mix that would achieve strengths close to the natural rock. The location and orientation of the extensometers ensured that the maximum amount of rock expansion was recorded, while the weak grout mix prevented the extensometer from creating a stiff column within the rock and giving falsely low readings. Combined with the target monitoring and inclinometer readings, the extensometers presented an excellent profile of the rock expansion beyond the shoring wall.

The monitoring program integrated the instruments to determine, with redundancy, the full extent of movement and behaviour of the shoring wall, soil and rock mass. The rates and magnitudes of movement were compared on a weekly basis with all three types of instrumentation to determine the validity of the results. The monitoring layout, showing the locations of inclinometers and extensometers, is shown in Figure 5.

6. MONITORING RESULTS

The monitoring program proved to be successful at providing early warning of problem areas at The Bow. It was used with efficiency to accommodate the Observational Method during construction to deal with issues that occurred, and to define the bedrock and shear band phenomena’s behaviour and movement extents.

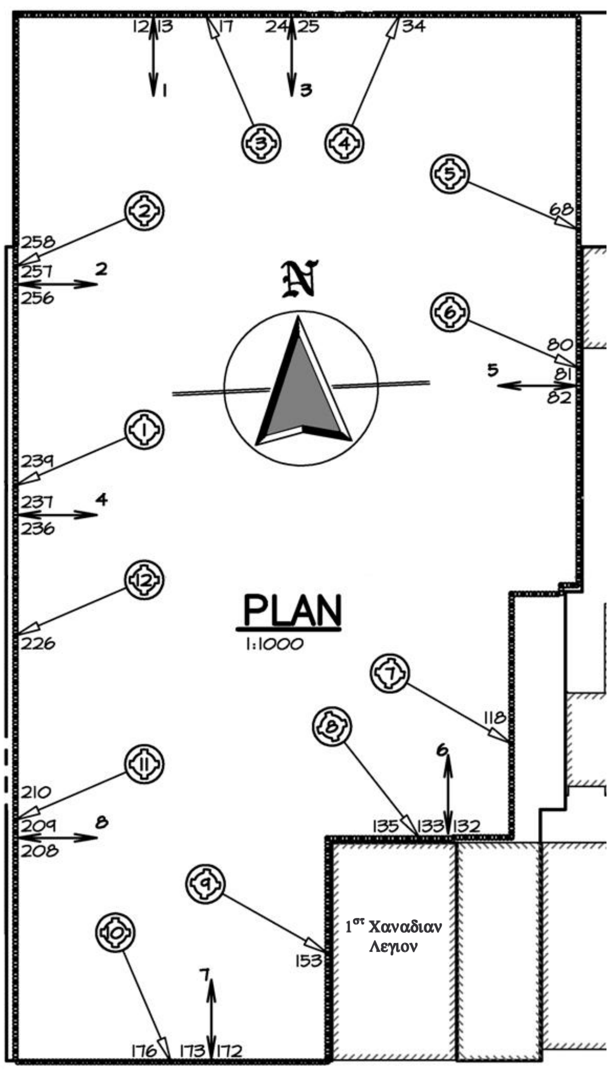

Figure 5. Inclinometer and extensometer locations

Monitoring proved the shear band phenomenon to be the driving force for large horizontal movement at the Bow. The inclinometer plots, verified by precision monitoring, show definitive deep shear band propagation in the north wall at Inclinometer #4 (Figure 6) as compared to normal rock expansion in Inclinometer #5 (Figure 7). In Figure 6, the shear band effect at elevation 1021 m accounts for over half of the total horizontal movement at the shoring wall. In addition, the movement trend above the shear band in Inclinometer #4 closely resembles the movement in Inclinometer #5. This comparison confirms that in-situ stresses alone were not the driving force of large horizontal movement at other excavations. It is important to note that the shear band phenomenon was manifested in the inclinometers at a very early stage in the excavation. Though the shear band existed in the rock at elevation 1021 m the shear band showed characteristic movement with excavation of just 8 to 10 m or to elevation 1036 to 1038 m. Thus the inclinometers provided an excellent early warning of the effect.

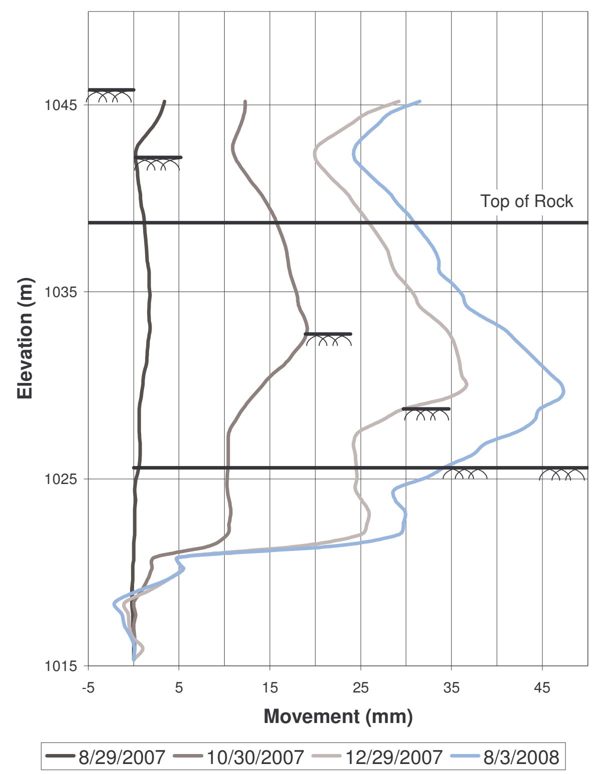

Figure 6. Readings from Inclinometer #4

Extensometer plots, as shown in Figure 8 of Extensometer #4, were used to confirm the creation of a rock mass to resist movement caused by rock expansion, as desired by the design. This plot indicates the relative movement of the rock mass up to 25 m from the face of the excavation. The extensometer plot shows that the rate of movement increases after the first two potentiometer nodes, or 8 m from the face. This distance corresponds with the anchor zone, showing that the tensioned anchors are creating a block of rock resisting expansion, just as the design required. Further modeling indicated that when using representative rock bulk moduli the movements could not be explained by shear band alone. The superposition of elevated K_0 values ($K_0 = 2.0$) was required in order to gain acceptable movement correlation.

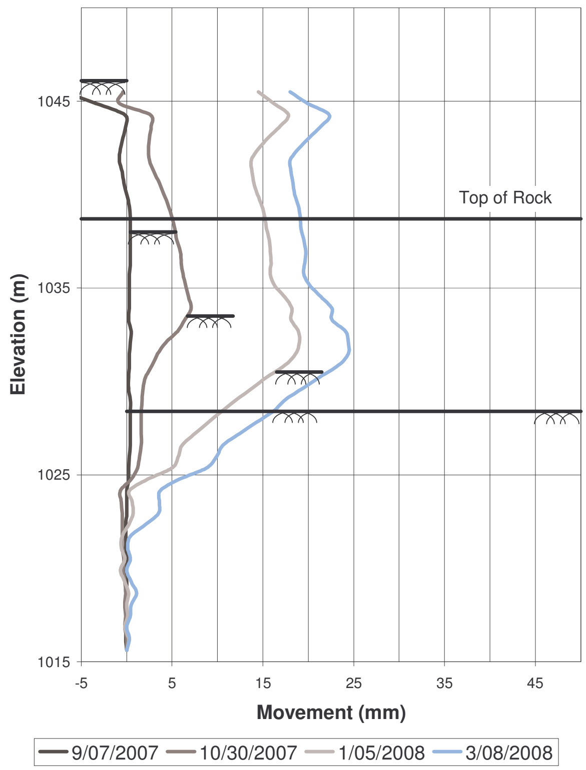

Figure 7. Readings from Inclinometer #5

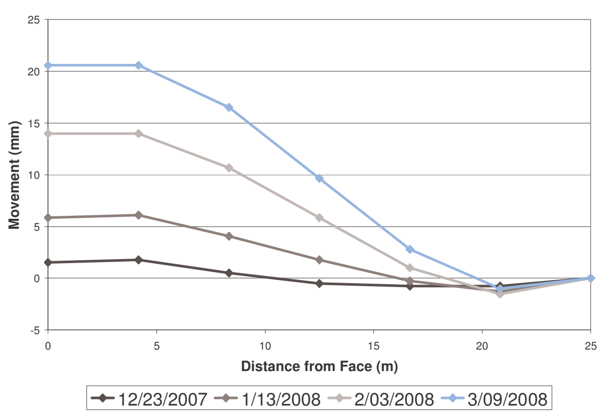

Figure 8. Readings from Extensometer #4

Shoring wall behaviour also confirmed the design predictions. When the shear band along the north wall was identified and indicated large horizontal movements, lift-off tests of available anchors were performed to confirm stresses. Anchors in the top two rows, located in the gravels on the caisson walls, only experienced an increase of 10 to 30% in load, while anchors located within the rock mass experienced up to 80% increase in loads. This confirmed that the upper sections of the soil and caisson wall would be little affected by rock expansion.

The analysis was used to back calculate the movements observed at site to refine the geotechnical parameters and build a more accurate model. The ability to match the movements as excavation proceeded and encompass the various geological scenarios proved important for prediction of the total expected movements as excavation proceeded to final grade. Table 2 shows the predicted and observed movement at the top of the shoring wall.

Table 2. Predicted and recorded movement at the top of the shoring wall.

	Predicted Movement at Top of Shoring Wall (mm)	Movement at Top of Inclinometer (mm)
No Shear Band (Incl. #5)	16 - 21	18
Shear Band (Incl. #4)	35 - 45	31

The FLAC model was continuously updated during construction of the shoring wall. As more information became available from inclinometers and extensometers, the FLAC model was refined to better aid in section specific issues.

The shear band was first observed along the north wall and the northwest corner of site. The results of Inclinometers #3 and 4 indicated a clear shear band forming at elevation 1021 m. When the shear band was first identified, the design/build team opted to increase the number of D Row anchors along the north wall in an attempt to increase the resistance to rock expansion. Upon further monitoring of the inclinometers, it was deemed that the additional anchors had little effect. Monitoring results from Extensometer #1, shown in Figure 9, indicate that rock expansion was occurring well past 25 m from the rock face.

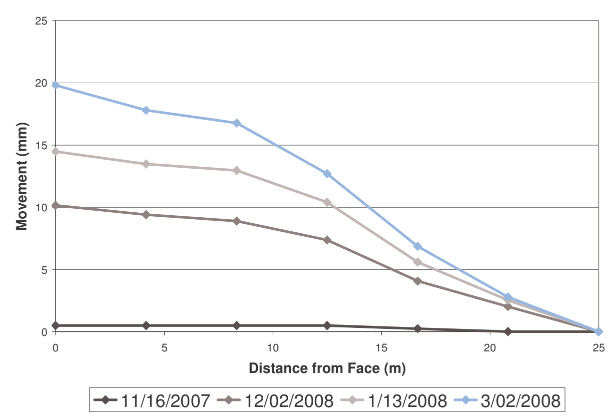

Figure 9. Readings from Extensometer #1

The area of effect caused by the shear band was both deeper than the anchors and extended back further than the proposed anchor lengths. Inclinometer and extensometer readings indicated that restraining movement due to deep shear band propagation would not have been possible using conventional anchors. Given that the movements were within the predicted amount, it was decided that no further attempts would be made to resist the movement, and that the wall would be continuously observed to ensure movement continued in the predicted manner.

During the accelerated shoring and excavation at 6th Avenue, a section of the east shoring wall by the Andrew Davison building experienced sudden movement into the site, occurring when excavation rates were up to twice as rapid as other areas. The result was a sudden movement into site of up to 40 mm. This movement occurred over the course of 3 days. Upon discovery of cracking at the ground surface and in the caisson wall, excavation was temporarily stopped while monitoring results were collected and analyzed. Precision target monitoring in the area was increased to twice daily. The results from the monitoring indicated that the movement was sudden and limited, allowing for the continuation of excavation in the area.

The work conducted to better understand the shear band effect proved fortuitous as the excavation progressed southward. At the location of the Regis Hotel and the Royal Canadian Legion in the southeast corner of the site another mobile shear band was observed, this time at elevation 1028 m, or 3 m above proposed final grade, as shown in Figure 10.

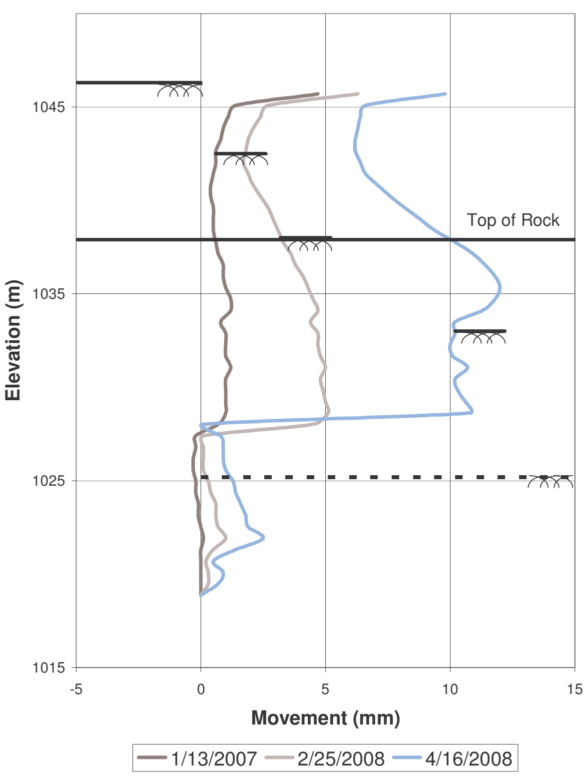

Figure 10. Readings from Inclinometer #8

Additional analysis indicated that the installation of additional anchorage through the shear band would partially mitigate the predicted increased movements. A program to triple the anchorage through the shear band was undertaken thus reducing the predicted increase in shoring wall face movements from 15 to 20 mm to 10 to 12 mm.

The early warning provided by the inclinometers enabled time for observation, modeling, discussion and reaction without interrupting the construction schedule.

7. CONCLUSIONS

Given the results of the monitoring program, the FLAC analysis for The Bow site accurately predicted the practical and geotechnical issues to ensure a proper excavation support system. The detailed background research and intensive FLAC predictions correctly identified the movement characteristics of the soil and rock mass. Monitoring further indicated that the design philosophy of creating a solid rock mass to act as a gravity dam resistance was properly executed. The caisson wall over shotcrete effectively negated the issues of soil loss, groundwater ingress, and bedrock degradation.

The monitoring program was effective in supporting the Observational Method, as well as identifying and measuring the shear band phenomenon. From the results of the program, the shear band effect has been identified as the driving factor for large horizontal deformations in deep, downtown Calgary excavations. Additionally, the monitoring program confirmed the assumptions and predictions of the finite element analysis, and was instrumental in aiding decision making during construction.

The extensive monitoring, correlated to FLAC analysis and re-analysis throughout the excavation program allowed the design team and owners to react to large observed shoring movements in a sensible, safe and economical manner that did not impact the construction schedule. In the absence of a correlated design and an analytical approach based upon effective monitoring, the client’s reaction to the large movements observed could have been unnecessary and wasteful.