A STUDY OF THE DEEP EXCAVATION INDUCED ROCK STRESS OF A PROJECT SITE

Zhaojun Xia, McClymont & Rak Engineers, Inc., Rexdale, Ontario
John Franklin, Franklin Geotechnical Engineering Ltd., Orangeville, Ontario
Juan Chahine, McClymont & Rak Engineers, Inc., Rexdale, Ontario
Bin Rao, McClymont & Rak Engineers, Inc., Rexdale, Ontario
John Dworatzek, McClymont & Rak Engineers, Inc., Rexdale, Ontario

ABSTRACT
The recognition of rock stress in Canada and its formation had been identified, discussed and well documented in previous investigations. There is, however, a relative lack of intensive study for the impacts of newly released rock stress, due to excavation, on the foundation design of engineering structures. The research, observations and monitoring results for the project of Canada Brick in the City of Burlington, Ontario, Canada, presented: The magnitude and directions of the rock stress on the project site; The potential detrimental effects of these high stresses on foundation design and engineering excavations; The necessity of essential time delay between rock excavation and lining placement to release rock stress; The special considerations on rock stress for foundation design and footing type selections. Theoretical analysis and practical computer modeling provided a quantitative consideration on the magnitude of rock stress, deformation/creep during and post excavation, and associate stress redistribution, footing deflection in the process of project structure design. In situ monitoring supplied the state of art information on site for the project design and construction.

RÉSUMÉ
La reconnaissance de contrainte de roche à Canada et sa formation a été identifiée, discutée et bien documentée dans des investigations précédentes. Il y en a une manque intensive pour les impacts des nouveaux contraintes de roche relâchées, à cause d'excavation, sur le concept des fondations des structures. Les recherches, observations et les résultats de surveillance au projet de Canada Brick à Burlington, Ontario, Canada ont présenté: La magnitude et directions du contrainte de roche dans le projet; L'effet adverse potentiel de ces contraintes élevées sur le concept de fondations et excavations; La nécessité d'une période d'attente essentielle entre l'excavation de roche et le placement du revêtement pour relâcher la contrainte de roche; Les considérations spéciales sur le fluage de roche pour le concept des fondations et la sélection de type de semelle. L'analyse théorique et modelage assisté par l'ordinateur ont fournis une considération quantitative sur la magnitude du contrainte de roche, la déformation pendant et après l'excavation, et la redistribution du contrainte associée, semelle déflection dans le cours ou progrès de construction. Surveillance en place a supplié l'information nécessaire pour les plans et la construction du projet.

1. INTRODUCTION

The subject site for Aldershot Plant of Canada Brick is located at the east of Hickory Lane and north of the North Service Road in the western portion of the City of Burlington, Ontario, Canada. The property is irregular in shape with an area about 16 hectares (40 acres). The site was vacant and covered by grown grass, brushes and trees. Surface was originally undulating and sloping gently south towards Lake Ontario.

The project consisted of: a main brick manufacturing plant which is 252 m long and 60 m wide, a brick storage yard, a stockpile area for material mixing, a parking lot, a shipping area and a road inside of the property (Figure 1).

The preliminary geotechnical investigation in the area of the proposed manufacturing plant was carried out in early May 1998. Results indicate that up to 15 m of weathered and sound sedimentary bedrock, Queenston Formation, was above the anticipated footing levels, see the contour map of Figure 2.

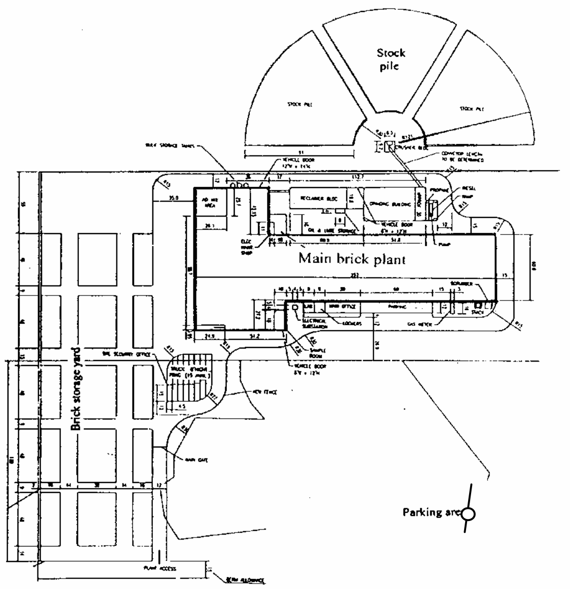

Figure 1. Plan of Aldershot Plant, Canada Brick

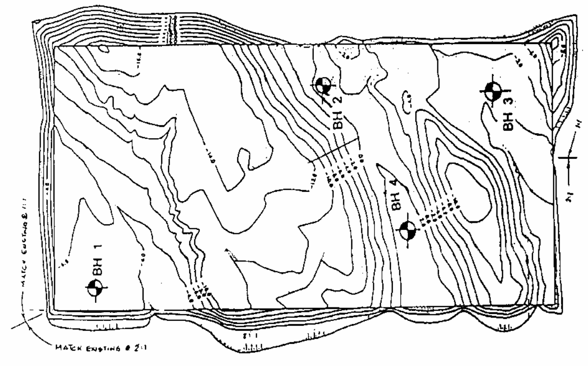

Figure 2. Contour map of site excavation

The primary calculations indicate that about 350,000 cubic meters, or 720,000 tons, of rock will be removed during the site preparation work. The planned removal of the bedrock mass will result in substantial changes in the in situ stresses, at the anticipated foundation levels.

The removal of the rock mass, in term of a unit bearing pressure, will result in about 360 kPa net pressure reduction below the foundation level. The underlying rock formation could be subject to heave, because of the quality of shale bedrock and the magnitude of the horizontal in situ stress.

2. OBJECTIVES

Our concern is the magnitude and the rate of dissipation of the potential rock heave and its impact on the slab on grade of the main manufacturing plant. The specific objectives of the study are to give emphasis to the following:

- To understand the properties of the Queenston shale bedrock formation, especially the magnitude/rate of potential heave, as they affect the design and construction of the proposed plant.
- The required design data and the impacts of the rock stress on the brick plant building foundations, excavation, utilities, slab on grade and pavement construction.
- Computer modeling to analyze the performance of the foundation of the brick plant building under impact of rock stress and in situ monitoring of the processes of rock heave and stress dissipation.

3. PRECONSTRUCTION FIELD INVESTIGATIONS

Fieldwork was carried out on May 22, August 21 and 22, 1998 and consisted of excavating 13 test pits and coring 4 boreholes. The boreholes were advanced into the bedrock by BQ size core bits. The approximate locations of the boreholes are shown on Figure 2. Four (4) test pits were excavated in the area of borehole No.1 and 3 test pits in the area of each of the remaining boreholes. Depths of test pits were in the range of 1.2 m to 5.0 m deep, and coring boreholes were advanced to 6.0 to 10.4 m measured from ground surface (Semaan and Rak).

3.1 Subsurface Conditions

The subsurface conditions encountered through test pits and coring boreholes at the project site are summarized as follows:

- ***Topsoil*** Loose or soft topsoil was encountered and ranged from 150 to 200 mm in thickness in the area of Borehole 1 and Borehole 3.
- ***Fill*** Soft to stiff clayey silt fill with fragments of broken shale and limestone slabs was encountered in all the test pits. The fill was grey to reddish brown, moist to wet and extended to the original clayey silt or weathered shale encountered at depths ranging from 0.6 m to 4.0 m.
- ***Clayey Silt*** Soft to hard clayey silt was encountered at the surface or below the fill in all the investigated area. The grey and wet clayey silt/silt till was moist and extended to weathered shale bedrock.
- ***Shale*** The sedimentary shale bedrock was encountered in all test pits and boreholes, and extended to the maximum depth of the investigation. Infrequent layers of limestone were encountered within the shale mass. The shale was highly weathered to weathered in the upper levels and becomes harder with depth.

3.2 Groundwater

Levels of groundwater were recorded during field investigation. Most of the test pits and boreholes remained dry on completion of field investigation. Locally perched water was encountered in the clayey silt layer below the fill.

4. BEDROCK FORMATION

The shale bedrock encountered in the project site belongs to the Queenston formation. Queenston shale consists of about 90% shale with occasional limestone layers (Franklin and Gruspier, 1983). The shale is calcareous, with harder and more durable shale bands parallel with the bedding and occasionally at right angles to form sedimentary dykes (Franklin and Gruspier, 1983).

Other characteristics include: thick bedded structure, red in color, with about 60% of clay minerals (only minor percentage of expansive clays), and relatively weak in the upper layer, about 1.5 to 3.0m in depth, caused by long term substantial weathering. Borehole core recoveries of

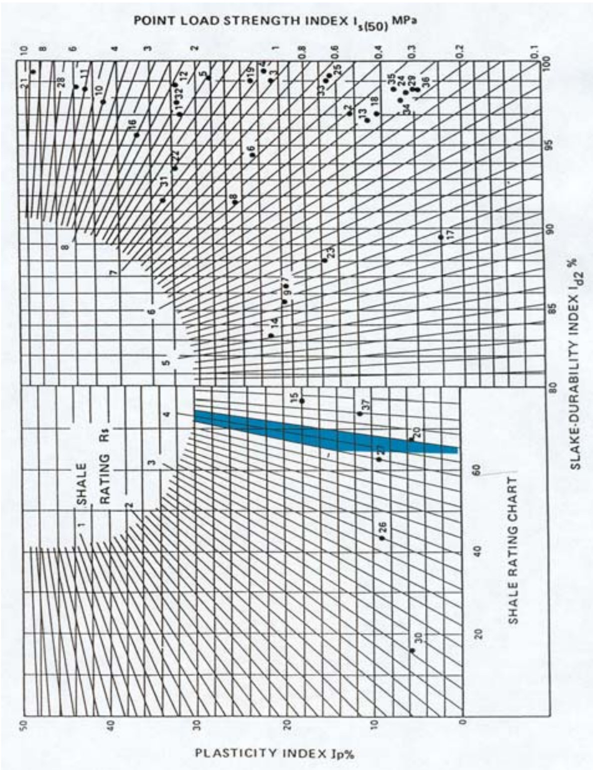

Figure 3. Shale rating chart from Franklin & Gruspier

the shale/limestone formation, below the proposed sub grade level, varied from 49% to 69% and the Rock Quality Designation (RQD) ranged from 19% to 68%. Based on these results, the in situ quality of the rock is defined as fair, except in the upper highly weathered zones where the rock is defined as of a poor quality.

The typical durability of the Queenston shale formation is I_{d2} = 64% (Franklin and Gruspier 1983). The laboratory tests carried out on selected core samples, representative of the encountered shale formation, indicated a Plasticity Index I_p ranging from 1.0 to 14.8%, with an average value of 7.9%.

Plotting the durability I_{d2} and the range of plastic index I_p of the Queenston shale on a shale rating chart, Figure 3 (Franklin and Gruspier 1983), we find out that the estimated shale rating is in the range of 3.9 to 4.1 with an average value of 4.0, which is relatively low tendency for settlement/heave.

5. HORIZONTAL ROCK STRESS

There are two basic classes of evidence (Franklin and gruspier 1983) for the existence of high horizontal ground stresses: the unusual and often expensive misbehavior of engineering works constructed in or upon highly stressed ground; and the occurrence, sometimes quite suddenly and violently, of post-pleistocene folds and faults in near surface foundations and quarry floors (White et al., 1973; Palmer & Lo, 1975). Reports and discussions of softer rock carrying high ground stress were appeared, long before the invention and development of the stress measurement techniques, in the technical literature as early as 1886 (Gilbert, 1886; Adams, 1927). Thereafter, the characteristics and behavior of high ground tresses and their detrimental effects on civil engineering, hydro power, transportation and mining structures is, historically, well documented (Gilbert, 1888; Adams, 1927; Lo & Morton, 1975).

Horizontal ground stress in the range of 5 to 15 MPa measured in shallow depth from 0 to 100 m have been reported in Canada. The two principal stress components of the horizontal stress are often similar in magnitude, with the major principal stress in a northeast to easterly direction (Franklin & Hunger, 1978). An empirical equation to define the average magnitude of theis horizontal ground stress has been suggested by Herget (1974):

$$\sigma_H = 8.16 + 0.04h \qquad [1]$$

where h is depth in meter and unit of σ_h is in MPa.

In the case of our project, to the 14 m excavation, the calculated maximum horizontal **s**tress in shale bedrock is about 8.72 MPa at northwest corner and mid east portion of the site.

6. DEEP EXCAVATION

The potential impact on the proposed base surface of the open cut excavation was evaluated by modelling the changes in stress-state and displacements of surrounding soil/rocks. The modelling was carried out by two-dimensional analysis, i.e. only the stresses and displacements in single plane were considered.

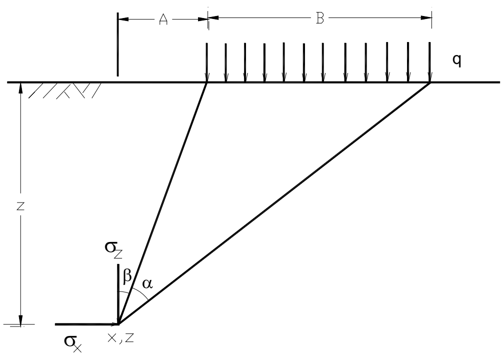

Figure 4. Stress due to the uniform pressure

The stress beneath the base of the open deep excavation was estimated with the basic elastic theory. The effect from the two sides of the unexcavated existing soil above the base plan of the open cut excavation can be modelled as a strip area carrying a uniform pressure (Graig, 1983). The stresses at point *x,z* due to uniform pressure *q* on a strip area of width *B* and infinite length are given in terms of the angles α and β defined in Fig. 4.

$$\sigma_z = \frac{q}{\pi}[\alpha + \sin\alpha \bullet \cos(\alpha + 2\beta)] \quad [2]$$

$$\sigma_x = \frac{q}{\pi}[\alpha - \sin\alpha \cdot \cos(\alpha + 2\beta)] \quad [3]$$

$$\tau_{xz} = \frac{q}{\pi}[\sin\alpha \bullet \sin(\alpha + 2\beta)] \quad [4]$$

The force generated due to direct excavation was assumed to be equal to the magnitude of the gravity force of the soil removed. The pressure from direct excavation can be expressed as

$$\sigma_g = h\gamma \quad [5]$$

where h is the depth of open excavation and γ is the average unit weight of soil and rock.

Considering the comprehensive effects of the ground stress, the stress from unexcavated side area and the stress generated by gravity force of the open cut excavation the combined total vertical stress (σ_v) and horizontal stress (σ_h) acting on the bottom of the excavation site and the surface of the side wall, respectively, can be expressed as:

$$\sigma_v = \sigma_z + \sigma_g + k_a\sigma_h \quad [6]$$

$$\sigma_h = \sigma_x + k_a\sigma_g + \sigma_H \quad [7]$$

where k_a is defined as the coefficient of active soil pressure under Rankine's state and theory. Previous experiment and study show that k_a is a function of soil internal friction angle ϕ and can be computed as

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} \quad [8]$$

Quantitatively, the computer modeled output of the potential maximum vertical and horizontal stresses are listed in Table 1:

Table 1. Stress development during excavation (kPa)

	Horizontal		Vertical	
	Theor.	Pract.	Theor.	Pract.
Gravity	111	111	390	390
Side	13	13	0.01	0.01
Rock stress	8760	600	2497	171
Total	8884	724	2887	561

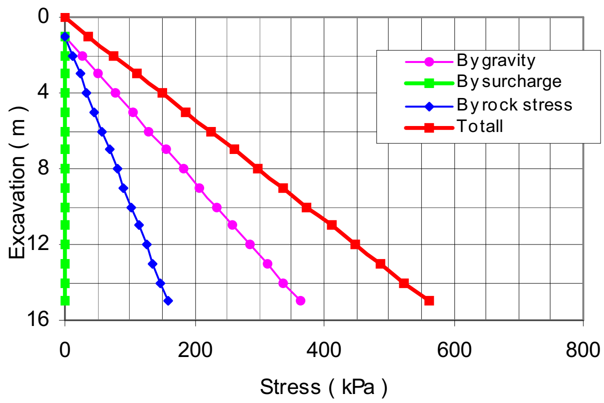

Figure 5. Computed vertical stress versus excavation development

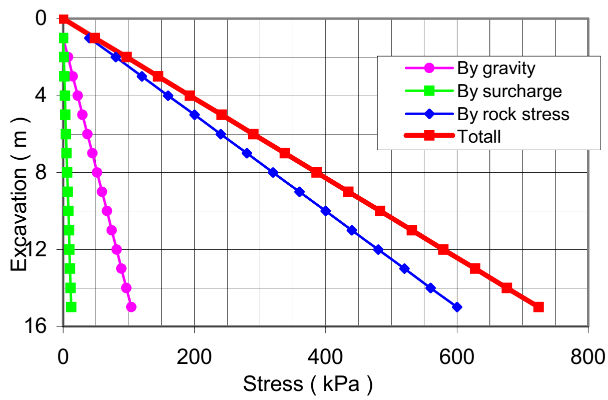

Figure 6. Computed horizontal stress versus

Figure 5 and 6 show that the computed vertical and horizontal stress development increase with the depth of the open cut excavation. The practical/differential vertical stress, which is the calculated difference from the initial day to the full completion of excavation is about 561 kPa in total. The vertical stress released from the gravity force of the rock mass removed is 390 kPa, which is 68% of the total pressure. The practical stress in horizontal direction is dominated by *ground/rock stress,* which is 600 kPa about 82% of the total stress 724 kPa.

The computed result shows that both in the vertical and horizontal direction, the stresses induced from the load of unexcavated side is limited, less than 2% of total amount, and can be negligible. Hypothetically, the stress released from the open cut excavation can reach 8884 kPa in horizontal and 2887 kPa in vertical direction. However, if we set up a base point of reference at the initiation of the site excavation, the relative results can be obtained from the differences of the stress/strain conditions between the final day and the starting day of the site excavation. It provides a relatively practical data, and commonly, this type-of-practical data is frequently used in planning and structural design.

7. ROCK MOVEMENT

The amount of rock heave up or lateral movement was estimated using elastic theory and corrected by a coefficient of time dependent creep/fatigue effect:

$$\Delta h = c \int_0^{2B} \varepsilon \cdot dz \qquad (9)$$

where Δh is the magnitude of rock heave; c is the coefficient of long term creep or fatigue effects, for short term excavation c = 1; B is the width of the site excavation, which is about 60 m; ε is the strain generated beneath the base surface of the deep open cut excavation and can be computed as

$$\varepsilon = \sigma / E \qquad (10)$$

where σ is the pressure generating rock heaving in vertical or horizontal directions and can be defined by equation (6) and (7) separately; E is the elastic modulus, for Queenstion formation shale is 0.9 Gpa measured in field with variations from 0.7 to 1.0 (Franklin and Gruspier 1983).

The computed results of rock heave at the center of the excavation site is about 74.8 mm in vertical, with 52 mm is generated from the stress release of the gravity force from the rock mass removal. The total horizontal deformation, approximately 99.6 mm, occured at about two thirds of the total excavation depth measured from the top. The movement caused by wetting expansion of the Queenston formation shale is not included. Practically, the magnitude of shale deformation caused by water intrusion or weathering exposure could be estimated, in engineering design, as a quarter of the total

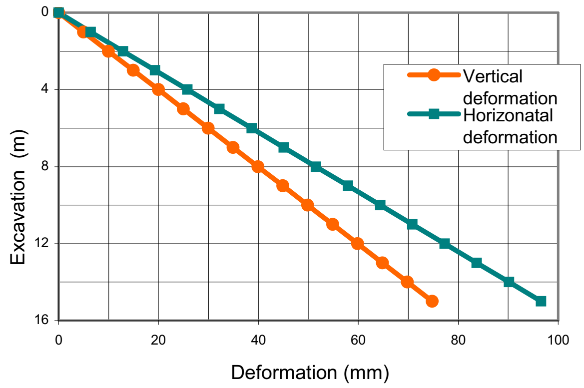

Figure 7. Computed deformation vs sit excavation

practical elastic deformation. According to the rock quality it may have certain amount of variances.

The modelled results of the amount of vertical heave up and horizontal deformation versus excavation development are shown in Figure 7. The horizontal displacement of 83 mm, approximately is about 86% of the total deformation, caused by the existing high ground stress releasing due to the rock mass removal is absolutely a dominant factor in the consideration of horizontal deformation, for details see Figure 8.

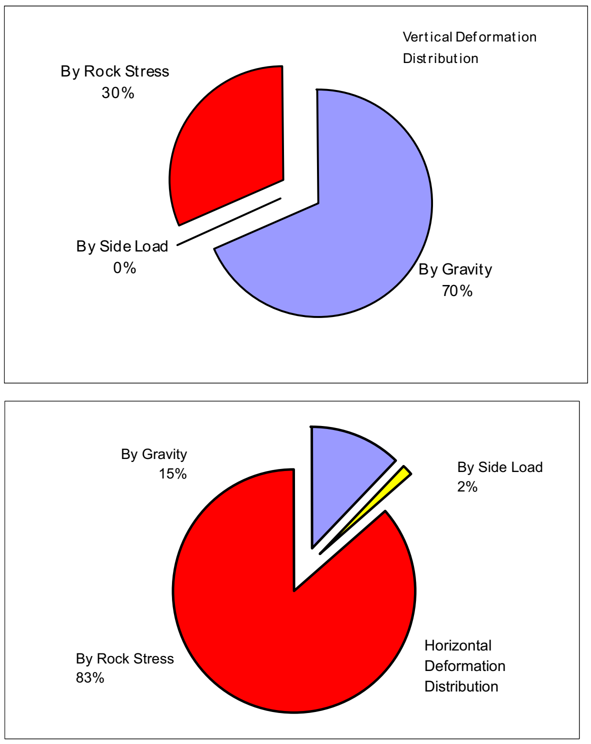

Figure 8. The percentage of vertical and horizontal deformation distribution

The computed vertical displacement and inward horizontal movement were compared with field measurements and monitoring data. Over 200 points in total were selected and used for site survey/monitoring. The collected data show that the calculated/modelled deformations are correlate with the on site observations, variance is in an acceptable range.

8. IMPACT MITIGATION

The computed stress and deformation, in both horizontal and vertical, on the critical points of the project excavation site are excessive. The vertical stress and deformation/heave up are functions of excavation depth. The horizontal stress and displacement are correlated to the magnitude of existing horizontal high ground stress. Therefore, the mitigation measures could be focused on the following approach:

- Release and reduce the quantity of existing high horizontal ground stress.
- Minimize the depth of strip layer of daily excavation to make enough time for ground stress dissipation.
- Select a suitable foundation type to enable the bearing of the potential ground stress and satisfy the deformation requirements of the brick plant.

Three systems/techniques were adopted to satisfy these three conditions. Digging a trench to release the existing high horizontal ground stress; introducing time lag and delay on schedule during and after excavation to dissipate vertical stress; and using a caisson-and-grade beam foundation system was found to be the best way and effective.

8.1 Excavate a Trench

To release the high horizontal ground stress a trench along with the deeper excavation side of the project site was excavated in advance. Bottom of the trench, about 1.5 m wide, is excavated to 2.4 m below the designed footing levels. The maximum cutting depth of the trench is about 17 m deep. Banks at both sides of the trench were constructed as 45^0 slopes in the sound shale – for detail see Figure 9. To ensure that the rock conditions underneath of the anticipated foundation level are undisturbed, the trench was kept at a distance of more than 1.5 m away from any footings of the building, measured from the top bank of the trench at the level of finished floor, e.g. elevation 132.00 m.

After completion of the brick plant construction, the trench was back filled with compact shale bedrock material. Care should be taken not to let the backfill material to become saturated in order to avoid frost heaving during the relatively long term winter months in Canada (Xia, 1993). At the bottom of the trench, water drainage system was installed, with weeping tile pipe and granular fill over 250 mm in depth

.

Figure 9. A cutting trench to release horizontal high ground stress

8.2 Designed time delay on schedule

To make enough time for bedrock shale releasing vertical stress, minimizing strip layer thickness and on scheduled time delay after excavation completion is an effective way. The objective is reducing the magnitude of deformation through the means of stress dissipation. Release stress progressively and remove certain amount of daily heave up during excavation to limit the final amount of deformation under control is also a successful way to reduce their adverse impact on project construction.

Figure 10. Monitoring points and scheduled delay

The designed construction schedule should, where possible, be adjusted to allow for several weeks, preferably months, delay between project full excavation and the starting of foundation construction. The scheduled time-hold up for the reported brick manufacturing plant was about 52 days. In this scheduled time period the observed maximum vertical movement was about 10 mm surveyed through 11 monitoring points – see Figure 10. To completely dissipate the internal stress and deformation caused by deep excavation a delay of 6 to 8 months, even longer, between the completion of the project site excavation and the starting constructions of foundations or the slab on grade, is anticipated.

8.3 Foundation structure alternatives

To limit the necessary dissipating time for shale heave up and reduce the waiting time for construction, structural alternatives should be considered. Tie down anchors and short caissons founded within the sound shale, was encouraged to be considered.

8.3.1 Tie down anchors

If a structure has to be built immediately after an open cut excavation, the designed time delay on schedule may not be valid. To over come the adverse effect of rock stress and deformation released by the open cut excavation, tie down anchors could be applied. The length of a tie down anchor normally is no less than 1.2 times of the depth of excavation.

Suggestions for the site of Canada Brick, the effective tie down anchor length may start at a depth of about 17 m below the anticipated foundation level and there should be virtually zero heave at the anticipated foundation level, if the proposed footings could be installed immediately upon the planned removal of the 15 m of the shale overburden (Semaan and Rak).

8.3.2 Short caisson footing

A short caisson/grade beam foundation system was recommended and provided for the structure. The caissons are designed as 0.91 m in diameter, 2.3 m in total length with reinforcing steel. Caissons are pre-augured and concreted immediately after drilling. The insertion depth is 1.5 m accordingly to the individual goetechnical conditions.

Caissons are designed to bear vertical and horizontal loads, shear force and over turning moment coming from super structure and the residual ground stress induced by the site deep excavation and the practical/differential pressures generated from existing high horizontal ground stress. Four (4) different cases of load combinations had been considered in mechanical analysis and computer aided Finite Element Modelling (FEM). The details of a load combination for caisson design are listed in Table 2.

Table2. Load combination of a individual caisson

	Load case	Vertical (kN)	Horizont (kN)	Moment (kN-m)
Central Columns	I	422.6	0.0	32.5
	II	- 221.3	0.0	16.3
Perimeter Columns	III	221.3	89.0	28.5
	IV	- 111.2	89.0	21.7

Note: - sign means force is in uplift direction

9. MONITORING AND OBSERVATIONS

Over 200 points in total were used as critical points of site survey and excavation control. Among them, 11 points were selected to monitor the vertical deformation of shale bedrock after completion of excavation. Extra care and protection were taken for the 11 equipments installed monitoring points. Plan locations and elevations of these 11 points are shown in Figure 9 and Figure 10. Elevations used in the drawings and tables are geodetic and survey was done by a licensed professional organization. Observation results of the on site monitoring are listed in Table 2.

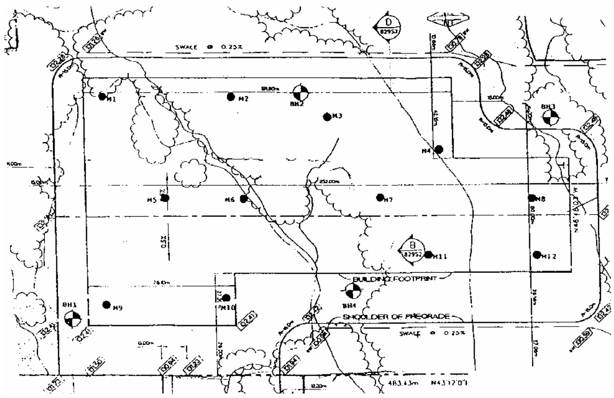

Figure 11. Plan of monitoring point locations

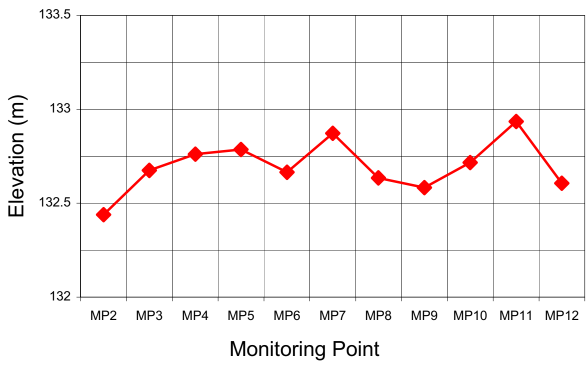

Figure 10. Elevations of selected monitoring points on shale bedrock heave up

The instrumentation of monitoring points consisted of installing steel rebar in pre-drilled holes. The borehole was 125 mm in diameter and extended to a depth of about 2.4 m below the existing surface, i.e. to the geodetic elevation of 130.3 m. The holes were backfilled with concrete. Vertical movements are being periodically monitored.

Table 3. Monitoring results of vertical movement

Date	1998		1999		Total (m)
	14/12	22/12	04/01	04/02	
MP-2	132.440	132.440	132.437	132.444	0.004
MP-3	132.675	132.675	132.672	132.679	0.004
MP-4	132.760	132.757	132.754	132.760	0.000
MP-5	132.786	132.788	132.784	132.793	0.007
MP-6	132.664	132.663	132.668	132.674	0.010
MP-7	132.871	132.871	132.868	132.874	0.003
MP-8	132.635	132.633	132.630	132.635	0.000
MP-9	132.584	132.580	132.581	132.586	0.002
MP-10	132.716	132.716	132.713	132.718	0.002
MP-11	132.934	132.936	132.932	132.938	0.004
MP-12	132.606	132.610	132.600	132.596	0.010

Monitoring started on December 14, 1998, which is the first day after the site excavation is completely finished and ended on February 4 of the year 1999, with total 4 observations in 3 periods of 52 days. The results/observations of the on site monitoring, during the on scheduled time lagging and delay, are listed in Table 3. The measured maximum magnitude of vertical heave up is about 10 mm happened on the monitoring points of MP-6 and MP-12. The rate of heave up was higher in the beginning and processed a progressively reducing in magnitude during the following monitoring period. The rate (mm/day) of maximum heave up was reducing with the time development of on scheduled time delay. The observed maximum daily rate of heave up was 0.5 mm/day in the first 8 days of early beginning, 0.24 mm/day in the following 13 days, and 0.17 mm/day in the finale 31 days. The observed magnitudes of actual heave up are shown in Figure 11.

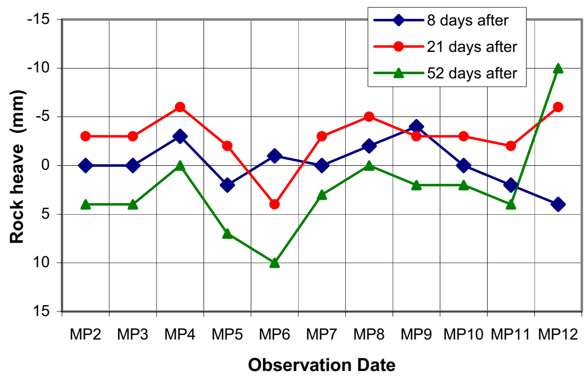

Figure 11. The magnitude of observed heave up during on scheduled time delay

10. DISCUSSION AND CONCLUSIONS

Rock stress and associated vertical heave up and lateral movement during open cut excavation have been observed on the project site, Aldershot Plant of Canada Brick, Burlington, Ontario, Canada.

Theoretically, the horizontal stress can reach 8884 kPa at 15 m deep open cut excavation and 8160 kPa at the ground surface computed according to the equation proposed by Herger et al. (1975). Practically, only the stress difference of the magnitude of 724 kPa was considered in the foundation design and construction of the Canada Brick.

The maximum heave up, for the red Queenston formation shale bedrock is about 74.8 mm in the centre of the site at the final stage of the open cut excavation whereas 96.6 mm horizontal movement was formed at the 2/3 of the cutting depth, measured from the top down to bottom, of the sidewall.

The magnitude of the vertical heave and lateral movement induced by the open cut to excavation is strictly determined by the type, the formation, the structure and the quality of the soft rock/shale. Mathematically, it is a function of the size, the depth and the amount of rock removal during the site open cut excavation and the existence of the high horizontal ground stress.

To mitigate the deterioration of ground stress and the heave or lateral movement of soft rock, the possibility of creep, the need for time-dependent analytical methods should be considered. If necessary to prevent/reduce the rock deterioration, trench excavation, tie down anchor and structural alternatives should be considered. To evaluate the actually heave up and horizontal displacement an on site monitoring work is encouraged to be considered and carried out.

The strategy, from a point of cost analysis, of designing/constructing a project in an area where high ground stresses are existing could be recommended as: to evaluate if the structure would be able to avoid their impact or the structure can be suited to their effect. If the above two conditions are inconsistent, the structure has to be designed strong enough to resist the adverse effect of the high ground stress.

11. ACKNOWLEDGEMENTS

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