# Heave Measurements within a Large Excavation

## Mesures du Soulèvement dans une Grande Excavation

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SYNOPSIS Field measurements of base heave in a large powerhouse excavation in glacial till and interglacial sand and gravel deposits are presented. To aid in the interpretation of field measurements, an elaborate simulation of stage excavation by finite elements was performed. The deformation modulus thus backfigured generally correlates with the design values previously determined in the field and laboratory testing of the subsoil. Predictions of foundation settlement are made on the basis of this correlation. Factors such as excavation geometry, soil properties, and the presence of shallow bedrock are examined, along with their impact on ground movement during excavation. A method of incorporating boundary stiffness terms to simulate the soil-rock interface in a finite element analysis is also described.

INTRODUCTION

The design and performance of turbomachinery foundations are to a large extent governed by settlement considerations. In the case of granular soils, the settlements are primarily of an elastic nature, and a knowledge of the deformation moduli is indispensable in the prediction and evaluation of foundation settlements. Hence, when Ontario Hydro's Wesleyville Generating Station was being planned to become a major power station with all of its heavy structures founded directly on granular till subsoil, a comprehensive field and laboratory testing program was carried out in 1972 to determine those deformation properties pertinent to design. The site, which is located approximately 60 miles east of Toronto (Figure 1), features some 40 to 45 ft of dense sandy silt till of glacial origin, overlying Trenton limestone bedrock and irregularly layered with an abundance of interglacial sand and gravel deposits (Smith, 1974 and 1975).

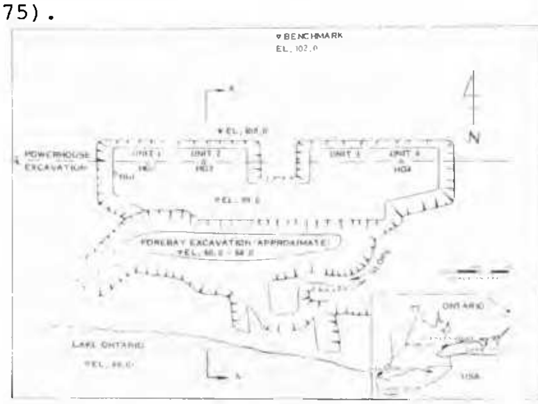


Fig. 1 Site layout and location of heave gauges in excavation

The soil testing program included in-situ pressuremeter tests, vertical and lateral plate-load tests, in-situ shear box tests and laboratory triaxial tests simulating the stress paths to be taken by the subsoil (Klym and Radhakrishna, 1972; Radhakrishna, 1972; Radhakrishna and Klym, 1974). Based on the results of these tests, design values of 1500 tsf and 750 tsf were respectively chosen as the deformation moduli of the dense glacial till and the interglacial sand and gravel deposits (Debidin, 1974). The field modulus was hence expected to vary between these two limits, depending on the relative proportion of the glacial till and the interglacial materials.

Construction planning of the Wesleyville site called for the founding of the powerhouse at a depth of approximately 19 ft below existing grade. A 1:1 excavation of this depth and having a length of 960 ft and a width of 100 ft was hence made to accommodate four turbine units, with a common central axis running in an east-west direction (Figure 1). Since measurements of excavation rebound could provide a verification of the deformation modulus of the subsoil and hence a better estimate of foundation settlement, a series of heave gauges were installed along the centre-line of the powerhouse excavation in June, 1975, prior to the commencement of excavation (Figure 1). This paper describes the field measurements thus obtained at various stages of excavation, and correlates these measurements with the results of a finite element simulation of the excavation process, giving implications on foundation settlement and ground movement during excavation.

HEAVE MEASUREMENTS

A total of four heave gauges were installed in the proposed powerhouse excavation between June 10 and June 13, 1975. One of these gauges (namely, HG1) was installed in a proposed discharge channel near the southwest corner of Turbine Unit No 1, while the three others were located at the centres of Turbine Units No 1, No 2 and No 4 respectively (Figure 1). The initial ground elevations at these points are given in Table I, along with the depths of the gauge tips below the original surface. To monitor the vertical movements of the gauges, a rock-anchored benchmark was installed remotely from the excavation, as shown in Figure 1. A sketch showing typical details of the heave gauges and the benchmark installed is given in Figure 2.

| DATE (1975) | GROUND ELEVATION (ft) | | | | HEAVE GAUGE ELEV (ft) | | | | DEPTH OF EXCAVATION (ft) | | | | HEAVE MEASURED (ft) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 |
| 16 JUN | 103.10 | 104.60 | 108.74 | 107.91 | 73.540 | 82.678 | 81.050 | 83.513 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 18 AUG | 98.74 | 98.83 | 98.74 | 98.71 | 73.543 | 82.682 | 81.061 | 83.061 | 4.36 | 5.57 | 10.00 | 9.20 | 0.003 | 0.004 | 0.011 | 0.023 |
| 16 SEP | 98.60 | 98.50 | 98.40 | 98.42 | 73.544 | 82.683 | 81.062 | 83.537 | 4.50 | 6.10 | 10.34 | 9.49 | 0.004 | 0.005 | 0.012 | 0.024 |
| 14 OCT | 96.10 | 97.90 | 96.90 | 93.70 | 73.546 | 82.685 | 81.063 | 83.540 | 7.00 | 6.70 | 11.84 | 14.21 | 0.006 | 0.007 | 0.013 | 0.027 |
| 24 OCT | 91.80 | 97.00 | 96.00 | 89.20 | 73.555 | 82.688 | 81.065 | 85.544 | 11.30 | 7.60 | 12.74 | 18.71 | 0.015 | 0.010 | 0.015 | 0.031 |
| 31 OCT | 91.80 | 97.00 | 96.00 | 89.20 | 73.554 | 82.688 | 81.064 | 83.543 | 11.30 | 7.60 | 12.74 | 18.71 | 0.014 | 0.010 | 0.014 | 0.030 |
| 4 DEC | 90.30 | 89.60 | 89.30 | 89.00 | 73.524 | 82.660 | 81.034 | 83.483 | 12.80 | 15.00 | 19.44 | 18.91 | 0.016 | 0.018 | 0.016 | 0.030 |

Table I Heave Measurements in Powerhouse Excavation

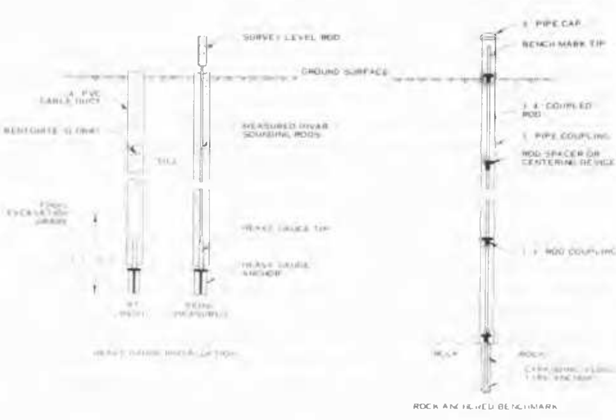

Fig. 2 Details of heave gauge and benchmark installation

For each gauge installation, a 4 in hole was first drilled to a predetermined depth, and a PVC cable duct was used as the casing. The hole was filled with a bentonite slurry to prevent piping of the sand and gravel deposits. The heave gauge was lowered into the hole and anchored into the undisturbed till at the bottom of the cased hole. For a heave measurement at any given stage of excavation, a 1/2 in diameter invar sounding rod of known length was lowered into the hole and allowed to rest on the top of the heave gauge. Levels were taken using a self-adjusting precise surveyor's level (reading to 0.001 ft), with reference to the rock-anchored benchmark which was located remotely from the excavation.

Table I summarizes the heave measurements taken between August and December, 1975, at various stages of excavation. Plots of heave measurements versus the depths of excavation at different times are given in Figure 3 for the four gauges. There is a good correlation between the amount of heave measured and the depth of excavation, suggesting that the rebound was generally of an elastic (or pseudo-elastic) nature. Moreover, the measurements apparently did not indicate any appreciable creep or time-dependent effect, at least not within the period of observation (August to December, 1975). With the coming of wintry weather in late December 1975, the readings were discontinued in view of the accumulation of snow and the possible effect of frost in the ground.

As of December, 1975, the depths of excavation at the four heave gauges (HG1 to HG4) were respectively 12.8 ft, 15 ft, 19.4 ft and 18.9 ft (Table I). At this stage of excavation, an upward movement of 0.36 in was observed in Turbine Unit 4 at gauge HG4, while the heave measurements obtained in the other units were considerably less, being in the neighbourhood of 0.2 in. Since the depths of the excavation at gauges HG3 and HG4 were both in the order of 19 ft, it appears that the discrepancy in heave measurements could not be fully attributed to the effect of the excavation depth.

Factors such as the variation of soil properties across the site, and the "berm effect" associated with excavated slopes were probably responsible for the discrepancy observed. Conceivably, the presence of a higher proportion of interglacial granular material would locally reduce the deformation modulus, and hence increase the amount of measurable heave. Nonetheless, the distribution of granular materials across the site appears to be so irregular as to render the analytical treatment of such a localized effect an impractical task.

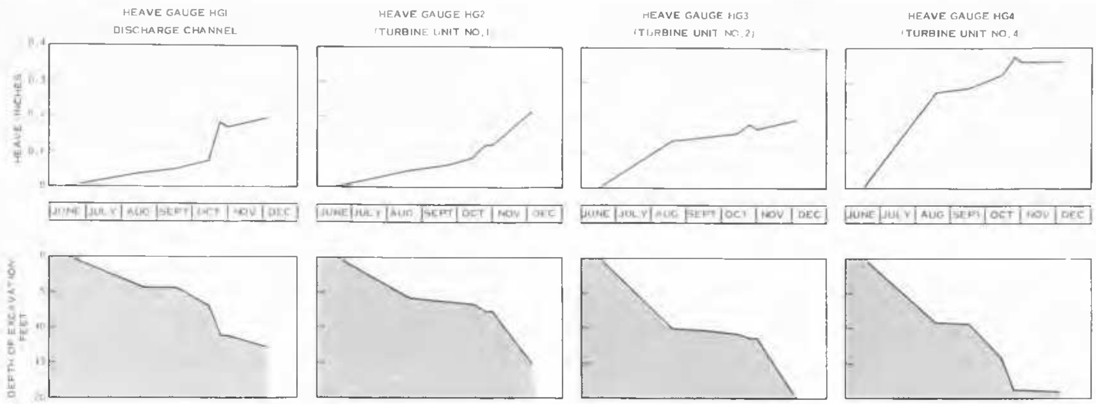


Fig. 3 Heave measurements at various stages of excavation

Referring back to the key plan shown in Figure 1, it can be seen that a considerable portion of the powerhouse excavation was actually linked to an adjacent excavation for the forebay structures. This latter excavation extended down for another 25 to 29 ft in depth to meet the surface of the Trenton limestone bedrock. Figure 4 illustrates these two excavations as they appeared in Section AA, which also contains heave gauge HG3. The forebay excavation produced a settlement of the bank, which in this case included the bottom of the powerhouse excavation (Figure 4). This "berm effect" would hence reduce the amount of measurable heave at gauges HG1, HG2 and HG3. On the other hand, since the excavation for Turbine Unit No 4 was relatively remote from the forebay excavation, the measurement of a larger heave there at HG4 is not inconsistent with the layout of excavations at the Wesleyville site. Thus, it appears that the forebay excavation is also an important factor to be considered in the interpretation of heave measurements.

solutions in the analysis of heave measurements. A better alternative in this case would be a numerical solution which can handle arbitrary geometry and the stage excavation process. The following section describes such a numerical simulation by the finite element method, primarily for the purpose of assisting in the interpretation of field measurements.

In general applications, a finite element simulation readily takes into account any stratigraphic variation in material properties. However, as earlier described, the highly irregular distribution of interglacial deposits across this site makes it rather difficult to assign elements systematically to represent a given soil type. It is, nevertheless, anticipated that the dense till and the interglacial deposits would have deformation moduli of 1500 tsf and 750 tsf respectively. Since the soil profile beneath any given point on the floor of the powerhouse excavation is a combination of these two soil types, the field modulus should

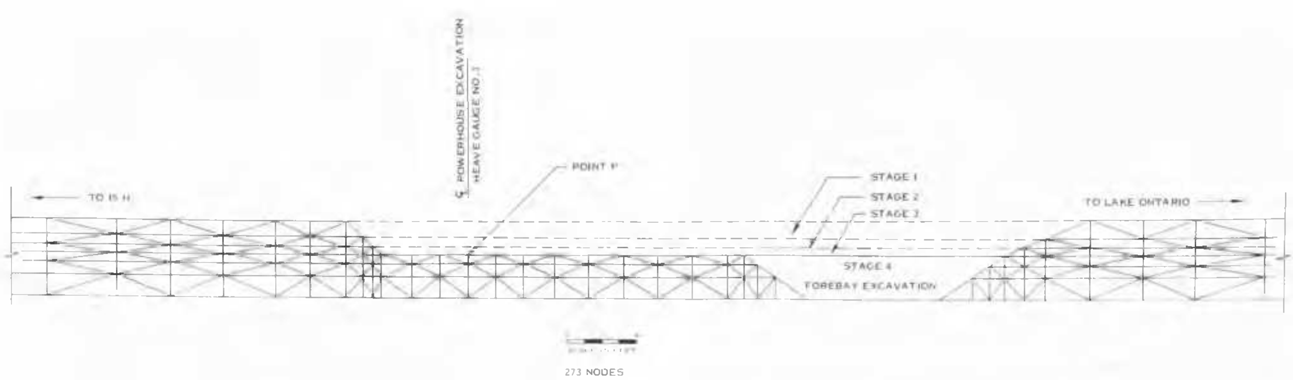


Fig. 4 Section AA and finite element mesh used in heave study

## ANALYSIS OF HEAVE MEASUREMENTS

The complexity of the excavation geometry, along with the "berm effect" associated with the forebay excavation, inevitably questions the validity of conventional mathematical

theoretically lie within these two limits. The objective of the finite element analysis is then to verify if the field measurements of heave fall within the range of values predicted by these two limiting moduli. If, on the other hand, the agreement between the design and the observed values is poor, an inference of the field modulus of deformation can still be made by backfiguring, and the settlements predicted for the turbine foundations accordingly revised.

Section AA in Figure 4 was thus arbitrarily chosen for the numerical study. A finite element mesh was generated, which accounts for four different stages of excavation* (Figure 4):

| STAGE | CUMULATIVE DEPTH OF EXCAVATION |
|---|---|
| 1 | 9 ft |
| 2 | 14 ft |
| 3 | 19 ft |
| 4 | Forebay Excavation to Bedrock |

Excavation is primarily an unloading process. In a finite element analysis, this is simulated by applying onto the excavated surface a system of forces equal and opposite to the initial stress system. During the stage excavation process, those soil elements located in the excavated portion of the continuum would be treated as "inactive" elements. In other words, they were assigned a nominal modulus such as 0.1 tsf, with the undesirably large movements at "inactive" nodes prevented by locking the latter in position.

A careful assessment of the boundary conditions was made in a preliminary study. The most significant boundary in this case is obviously the presence of bedrock at a shallow depth. In conventional applications, the boundary between soil and bedrock is often treated as a fully rigid boundary. However, it was found from the results of the preliminary study that such a treatment would induce a high degree of artificial constraint in the event of shallow bedrock, resulting in rather unrealistic stress conditions.

A research study was hence made to evolve a more realistic simulation of the boundary between soil and bedrock. A concept of replacing this boundary by a system of elastic springs was developed. With this approach, the bedrock would contribute stiffness components to the boundary nodes in accordance with its modulus of deformation and its area of influence. The load-deformation relationship of the bedrock at each boundary node was first determined by an elastic solution due to Giroud (1968). The stiffness contributions from the bedrock could then be computed and superimposed onto those boundary stiffness terms due to the soil mass. This gave stiffness contributions from the bedrock which were physically compatible with its deformation modulus, without invoking unrealistically large constraints at the soil-rock interface. Similar incorporation of boundary stiffness terms into finite element simulation has been adopted by Law (1976), in connection with the design of a loading platen for testing of frozen soils.

Besides the deformation modulus E, the other input variables required in the finite element analysis are the Poisson's ratio $\nu$, and the coefficient of earth pressure at rest K. A Poisson's ratio of 0.4 had been recommended for design, on account of the granular nature of the till and the interglacial deposits (Debidin, 1974). A coefficient of lateral pressure of similar value (ie 0.4) was also used in the analysis, to account for the effect of prolonged stress relief near the shoreline of Lake Ontario, and the cemented nature of the dense glacial till. The effects of Poisson's ratio and the lateral pressure coefficient on the results of the analysis were examined in some detail in the numerical study, and were found to be generally not excessive.

Figure 5 shows the upward movements computed for a point P, located on the floor of the 19 ft deep powerhouse excavation and directly above gauge HG3 in Section AA (Figure 4). The vertical movement of this point during the various stages of excavation are compared with the field measurements of heave in Figure 5. As the depth of excavation increases, there is computed a fairly linear increase in the upward movement. For a deformation modulus of 750 tsf, the predicted upward movement at this point amounts to 0.4 in when the excavation reaches a uniform depth of 19 ft (Stage 3). This drops off to approximately 0.3 in at the end of Stage 4, when the forebay excavation was completed. These figures clearly illustrate the effect of the forebay excavation on the amount of measurable heave.

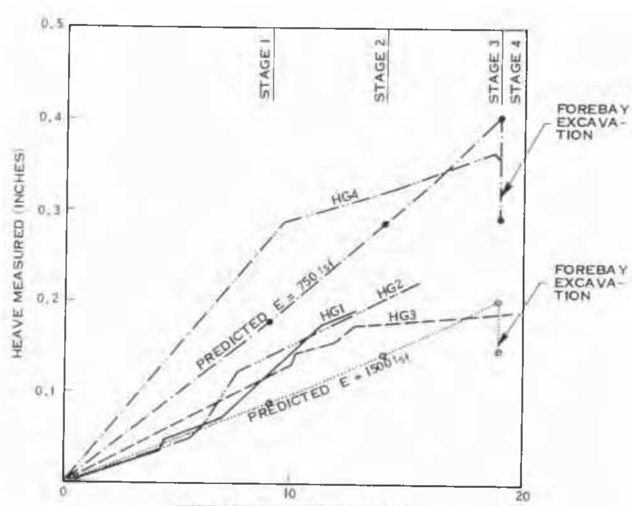


Fig. 5 Measured and predicted amounts of heave (predicted values refer to point P in Figure 4)

* In the field, the forebay excavation actually proceeded simultaneously with the powerhouse excavation. A simplication of the construction history had been made in order to delineate the effect of the forebay excavation.

On comparing the measured and predicted values of heave shown in Figure 5, it is obvious that the predicted values are generally in the same order of magnitude as those measured. It should, however, be noted that, at the final stage of excavation, the heave gauges were embedded at depths between 5.5 ft and 16.5 ft below the floor of the powerhouse excavation (5.5 to 8.3 ft for gauges HG2, HG3 and HG4). This implies that the heave of the floor could have been somewhat larger than those at the tips of the gauges.

For heave gauge HG3, which was embedded at a depth of 8.3 ft below the floor of the powerhouse excavation, the results of the finite element study indicate that the heave of the floor is approximately 30% more than that at the gauge tip. Taking this into consideration, it is apparent from Figure 5 that the field modulus is far closer to 750 tsf, than to the design value of 1500 tsf for the dense till.

Of the four gauges installed, the influence of the forebay excavation is probably minimal on gauge HG4 in Turbine Unit No 4 (Figure 1), on account of its remoteness. Theoretically, the amount of heave predicted for point P at the end of three stages of excavation (ie when excavated to a uniform depth of 19 ft) should approximate to the vertical movement measured at HG4. This probably accounts for the good agreement between the predicted value of 0.4 in and the measured value of 0.36 in given by gauge HG4.

Figure 6 shows the computed patterns of movements in Section AA at Stages 3 and 4. Besides the vertical movements, there is a very pronounced tendency for the soil mass to move horizontally, presumably as a result of the geometric conditions at this site. At Stage 3, the horizontal movement decreases from a maximum of 1.5 in near the toe to practically zero near the centre of the entire excavation. At Stage 4, as much as 3 in of horizontal movement towards the forebay excavation was computed.

In the field, the surveying staff had noted offsets generally in the order of "a couple of inches" towards the forebay excavation while using a transit to locate the heave gauges. This is apparently in qualitative agreement with the prediction of fairly large horizontal movements in that particular direction.

From the foregoing discussions, it is apparent that the deformation modulus of the subsoil at the Wesleyville site is probably in the order of 750 tsf. This is the design value chosen for the interglacial material, and half of that chosen for the dense glacial till. As earlier stated, these design parameters had been chosen on the basis of a comprehensive field and laboratory testing program. The discrepancy in this case between the experimentally determined moduli and the backfigured field modulus should be considered small for practical purposes. Heave measurements reported elsewhere, particularly those involving excavations in clay, have indicated much larger discrepancy between the field and laboratory moduli (eg Hanna and Adams, 1968; De Jong and Morgenstern, 1973; Serota and Jennings, 1959). The good agreement in this case is largely due to the low sensitivity of the dense till to disturbance, and partly due to the comprehensive testing done prior to design.

An important application of the results of heave measurements has always been the prediction of elastic settlements for foundation structures. It is understood that the turbine foundations proposed for Wesleyville GS are designed to carry a uniformly distributed load of 2.8 tsf. With a field modulus of 750 tsf, an elastic settlement in the order of 0.8 in is to be expected on these turbine foundation blocks. The maximum differential settlement resulting from

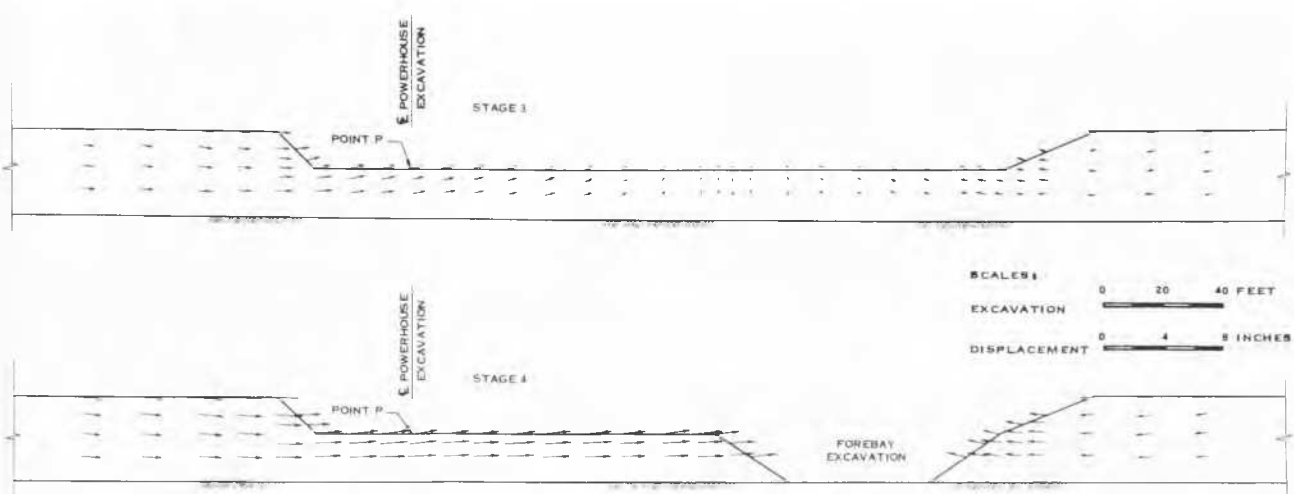


Fig. 6 Predicted patterns of displacement in excavation

non-uniform soil conditions is probably in the order of 0.4 in. Evidently, additional differential settlement may also occur as a consequence of non-uniform loading.

## CONCLUSIONS

Based on the field measurements obtained in the powerhouse excavation of the Wesleyville Generating Station, and their subsequent analysis, the following conclusions may be drawn:

1. The amount of basal heave of the excavation was measurable, despite the subsoil being a dense glacial till and dense interglacial sand and gravel deposits. The movements were primarily of an elastic nature, and correlated with the depth of excavation.

2. The deformation modulus backfigured from a finite element analysis compared favourably with the design values determined in soil testing. The good agreement in this case is probably due to the low sensitivity of the subsoil to disturbance, as well as to the comprehensive field and laboratory testing done prior to design.

3. The presence of neighbouring excavations, and that of shallow bedrock in the vicinity, both have significant effects on the amount of heave measurable. Such effects are best evaluated by means of a numerical method.

4. A method of incorporating boundary stiffness terms into finite element analyses to simulate bedrock is proposed. The addition of these stiffness terms, which are based on the load-deformation relationship of the bedrock at the soil-rock interface, gives boundary conditions which are more compatible with the mechanical properties of the rock.

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