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Soil behavior due to freezing and frozen column during excavation of soft ground

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ABSTRACT: This paper describes a case study of soft soil behavior due to freezing. Soft clayey ground was to be supported by frozen soil columns during excavation. Measured data of earth and pore water pressure at non-frozen zone were analyzed to obtain the enlargement process of the yielding zone around the freezing pipe. General geotechnical work during freezing, excavation, and thawing process were also introduced. The performance of frozen pile as retaining as well as under-supporting structure during excavation was shown based upon ground response of earth pressure, porewater pressure, and horizontal/vertical displacement.

1 INTRODUCTION

Ground freezing technique is sometimes very useful and even attractive not only to improve soft soil ground but also to depend on the unique characteristics of very high strength of frozen soil. with self healing nature. However, the soil behavior during freezing was not completely understood in terms of modern soil mechanics. The authors had a chance to monitor the response of soft clayey soils during freezing in Osaka.

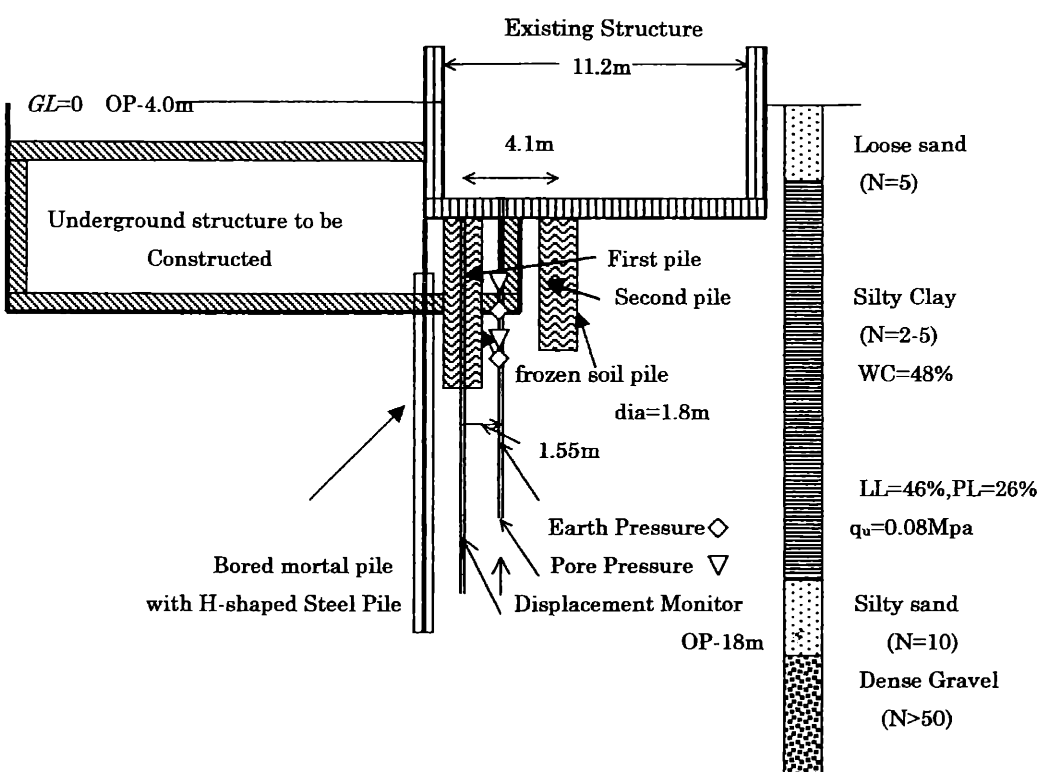

Figure 1. Geotechnical work of frozen soil column.

It is well understood that the frozen area is computed by temperature decrease in the soils and the expansion of frozen front with time based upon thermodynamics which was confirmed by in-situ measurement of soil temperature (Khakinov(1966)). Takashi(1971) showed theoretical consideration on the development of yielding of soils due to the freezing. The authors applied his theory to the measured behavior of soft clayey ground and presented some discussions on the results in this paper.

2 SITE CONDITION

2.1 *Geotechnical condition*

The top surface ground consists of soft alluvium clay layer (SPT N=2-3) of 18m in thickness from surface(GL=O.P.-4m). The lowest portion of the alluvium layer of about 5m changes from silty clayey to silty sand/sand layer(N=5-10) and continues to the dense gravelly sand Pleistocene deposits(N=>50) of about 6m in thickness.

2.2 *Geotechnical work*

The required geotechnical work was to extend underground structure horizontally preserving the existing structure, which was designed floating type and built on flat base foundation. It was planned to excavate the ground after freezing soil wall underneath the present structure to support and retaining the back ground during excavation as shown in Figure 1. Two lines of frozen columns were planned to support and to retain the existing structures during the excavation. The freezing process was divided into two stages of the first and second columns. The excavation was also divided into two stages, the first stage was to excavate the front portion of the present structure which was supported by the first frozen columns. After the front excavation, the bore-hole cement mixed column with H-shaped steel pipes were installed to underpinning the structure. The second stage was to excavate the underground space beneath the existing structure including the first frozen column.

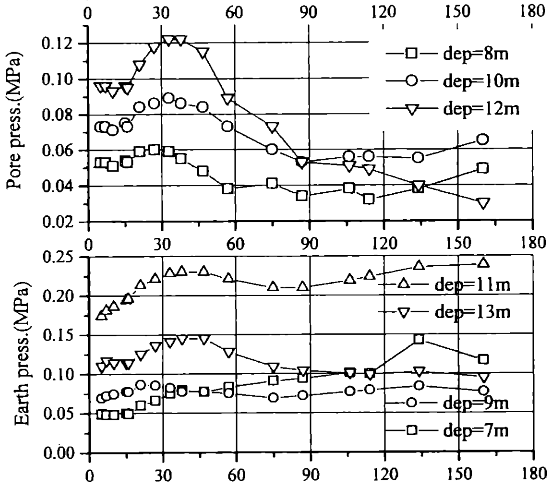

Figure 2. Change of horizontal displacement, earth, and pore water pressure at 1.55m from freezing pipe

3 OBSERVATIONAL PROCEDURE

3.1 *Monitoring ground*

To confirm the safety against instabilities against bearing capacity and sliding failure, observational procedures were introduced in the whole process of this geotechnical works. During the freezing process, P-wave velocity measurements were carried out to evaluate the frozen states of the soils. The behavior of soils around frozen column were monitored by measuring horizontal earth pressure, pore water pressure, horizontal displacement, vertical settlement, and differential settlements of the existing structure.

3.2 *Freezing soil*

Two lines of frozen soil walls were designed underneath the existing structure. Each wall was to be constructed by a line of frozen columns, each of which was frozen by a independently controlled freezing pipe with a distance of about 1.6m. After the start of freezing, the monitoring of horizontal displacements and stress, as well as pore water pressure at several depths had shown instantaneous response at the distance of 1.55m from the freezing pipe. Figure 2 shows time change of horizontal displacement, horizontal earth, and pore water pressure.

Figure3 shows the relation of earth pressure and displacement at the distance of 1.55m from freezing pipe. The estimated frozen radius increasing with time is also shown. The initial phase of the freezing

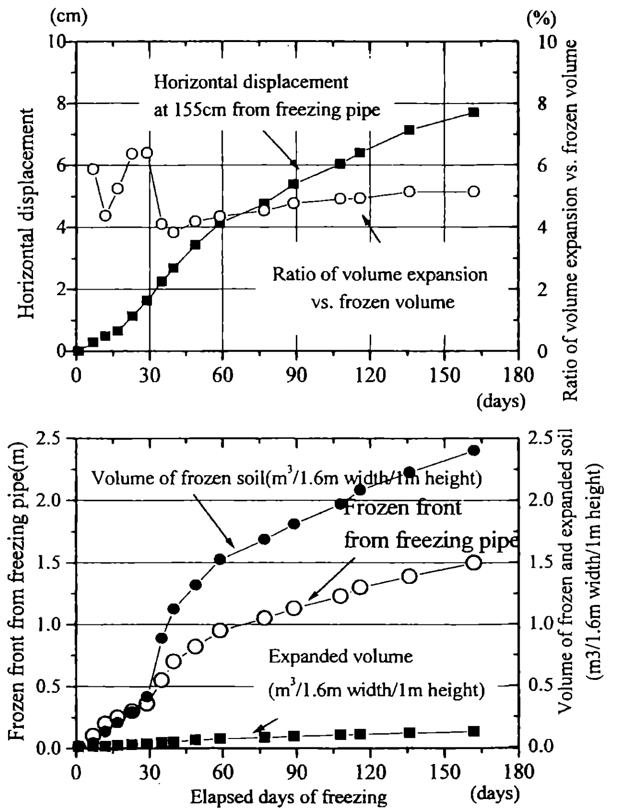

Figure 3. Frozen front with time and ratio of expanded soil vs. frozen soil in volume.

stage of the radius smaller than r=0.3m may be considered as cylindrical freezing.

The volume expansion due to freezing is characterized by freezing expansion ratio ξ:

$$\xi=\xi_0+c/\sigma \quad (\xi_0,\ c;\ \text{constant}) \tag{1}$$

where σ =confining stress(Mpa)

The parameter at the site was obtained as $\xi_0=3.2$, $c=0.52$.

In the Fig.3, the ratio of volume expanded vs. frozen volume is shown as about 5%, which is a little less than the expansion ratio $\xi=6.7$ given by eq (1) at the $\sigma=0.15$

3.3 *Stress change due to freezing*

The clayey ground around the freezing pipe is assumed fully saturated may be divided into three zones of frozen, plastic, and elastic state as proposed by Takashi(1971).

The stress is expressed in axial coordinate taking the axis at the center of the freezing pipe.

Principal stresses are denoted as σ_θ tangential stress and σ_r radius stress. The initial stress in the ground is assumed as homogeneous $\sigma_r=\sigma_\theta$. Soil is frozen from the boundary of freezing pipe and the frozen soil expands outwards.

The outward displacement by freezing expansion causes stress increase in radius component $d\sigma_i$ at r_i in the outside zone.

The radius stress increases by ($d\sigma_r$) and the tangential stress decreases by ($d\sigma_\theta$) with decrease with square of the radius in the elastic zone.

$$d\sigma_r = d\sigma_i (r_i/r)^2 \tag{2}$$

$$d\sigma_\theta = -d\sigma_i (r_i/r)^2 \tag{3}$$

If the stress difference ($d\sigma_r-d\sigma_\theta$) becomes greater than uniaxial compression strength q_u, the soil becomes plastic yielding state. The stress condition at the plastic boundary is

$$q_u = d\sigma_r - d\sigma_\theta \tag{4}$$

At the boundary (r_p)of plastic and elastic zone, $d\sigma_r=0.5q_u$ and $d\sigma_\theta=-0.5q_u$, eqs-2 and -3 become

$$d\sigma_i = 0.5q_u (r_p/r_i)^2 \tag{5}$$

$$d\sigma_\theta = -0.5q_u (r_p/r_i)^2 \tag{6}$$

Within the plastic zone the stress at r and increase of radius stress at r_f may be expressed by,

$$d\sigma_r = d\sigma_i - 0.5q_u \log(r/r_i) \tag{7}$$

$$d\sigma_\theta = d\sigma_i - q_u - 0.5q_u \log(r/r_i) \tag{8}$$

If r_f is chosen as r_p, eqs–7 and –8 become

$$d\sigma_r = 0.5q_u - 0.5q_u \log(r/r_p) \tag{9}$$

$$d\sigma_\theta = 0.5q_u - q_u - 0.5q_u \log(r/r_p) \tag{10}$$

3.4 *Distribution of pressure estimated by measured data*

If the unconfined compression strength of the clayey ground and increase of radius component of earth pressure at a radius r_m is known, stress state around the freezing pipe may be estimated as follows,

First, the radius of plastic zone(r_p) is estimated by eq(5) or eq(9).

The measured point is not known within the elastic or plastic zone. Which equation to be used depends upon the values of r_p and r_i in relation with elastic or plastic state. In generally, any point r is smaller than r_p ($r<r_p$) under plastic state and is larger than r_p ($r_p<r$) under elastic state.

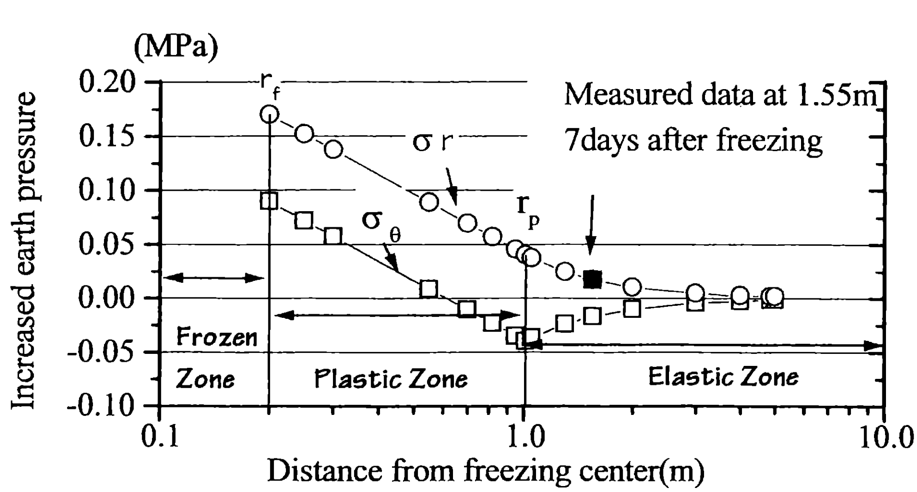

Figure 4. Stress distribution in soils surrounding.

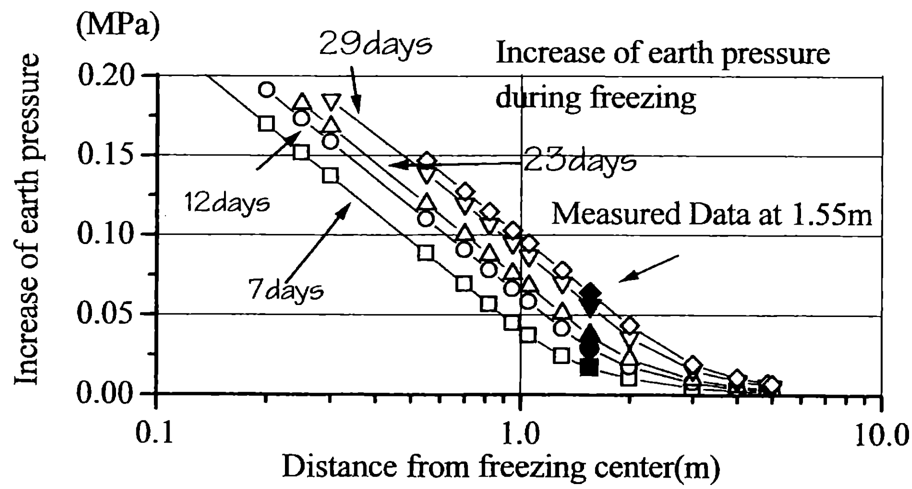

Figure 5. Time change of stress distribution.

If r_p obtained by eq(5) is less than the r_i, select this r_p as the correct value. The same procedure is applied for eq(9). If r_p from eq(9) is smaller than r_f, ignore this solution.

Once the radius of plastic zone is known, stress distribution of the elastic and plastic zones are easily evaluated based upon eqs (5), (6), (9), and (10) .The averaged unconfined compression strength at the site was q_u=0.08MPa. As an example, the stress distribution was shown based upon the measured field stress measured 7days after freezing at a depth of OD-11.2m with a radius of r_f=1.55m as a typical data shown in Figure 4. After 7days freezing, the expected radius of frozen soil is only r_f=10cm, the plastic zone is estimated at radius of about 1m(r_p=101cm). The stress at the boundary of frozen zone is obtained as $d\sigma_r$ =0.17MPa and $d\sigma_{\square}$ =0.09Mpa. The measured stress increased with elapsed time after freezing. The plastic zone also expanded to r_p=196cm after 29days of freezing when the frozen radius increased to r_f =36cm. After 29days of freezing, the fronts from the adjacent freezing pipe became to meet together and can not be considered as an independent circular configuration. The frozen front became more or less flat and the freezing situation is considered as plain strain condition.

At around 40days after freezing, the measured pressure had increased to nearly 0.8kg/cm^2, which is the same order of uniaxial compression strength. The stress state of the soil element under monitoring was compression and tension in radius and tangential component subjected to circular expansion during the initial step of freezing under elastic state.

After entering into plastic zone, the both stresses of radius and tangential component become increased keeping the stress difference constant as q_u. After the condition becomes plain strain, the expansion of tangential direction becomes constrained. The porewater pressure was also monitored and shown in

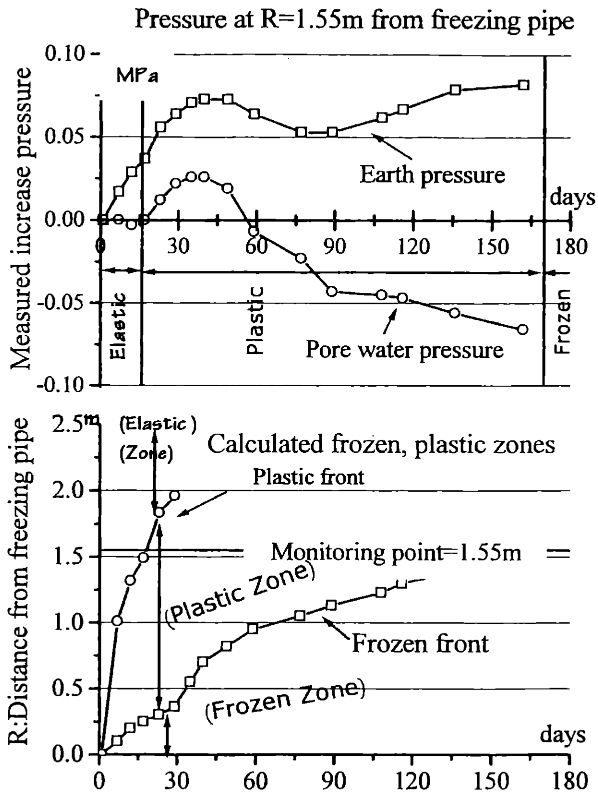

Figure 6. Expansion of plastic zone with time in connection with earth and pore water pressure.

Fig.6. The pore water was found to keep the initial value, which may correspond with the keeping the total mean stress as constant under the elastic state.

At around 25days when the plastic zone expanded to the monitoring point at r=155cm, the pore water pressure became increased, which is a response of increase of the mean total stress under plastic zone. After 40days, the pore water pressure was found decreasing. The decrease of pore water pressure may be due to suction effect from frozen front to the monitoring point.

The horizontal displacement was found 4cm at the maximum pressure. Since the soil between the first and second freezing column is constrained within the width of about 3m. The horizontal compression strain may be around 1.3% at pressure maximum to 3% at final stage. The soil might be said under completely yielded state. Figures 7. and 8. show the horizontal and vertical displacement according to the construction process. The maximum horizontal displacement of the ground was about 20mm at the depth of 12m. The building has settled about 20mm during excavation in front of the first frozen column. After installing the bored soil cement pile with H-shape steel pipe, the second step excavation was performed. No extreme deformation was monitored

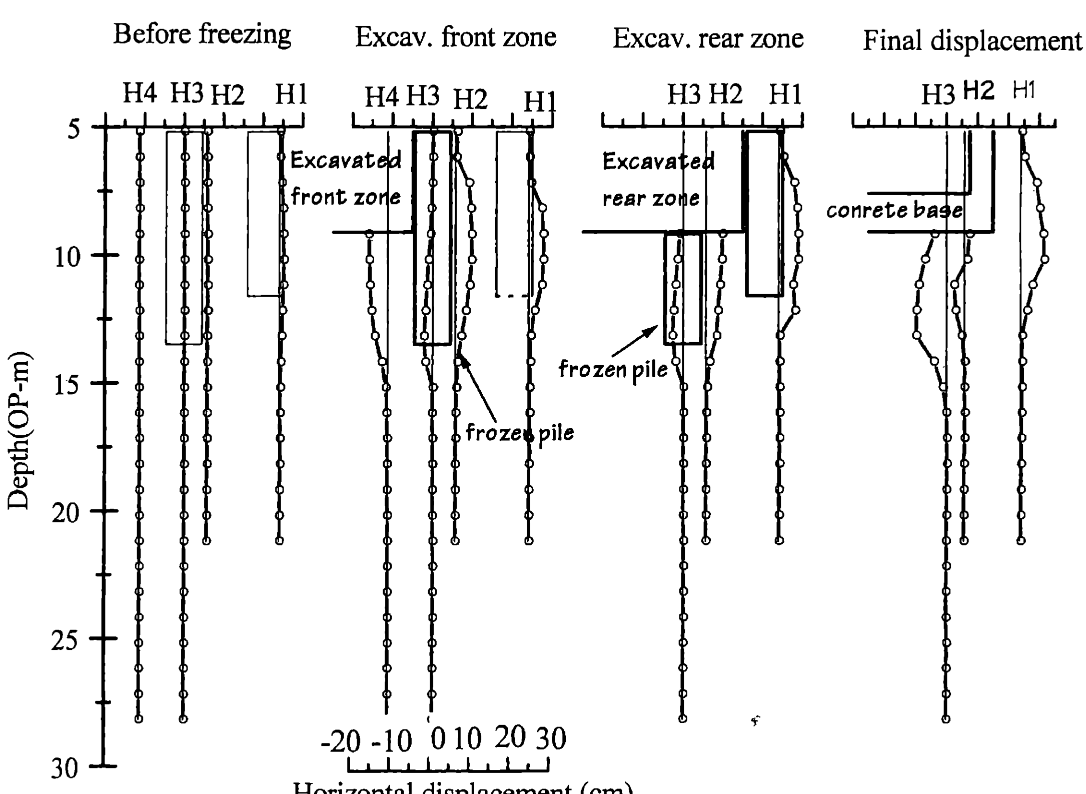

Figure 7. Horizontal displacement during the freezing, excavation, and thawing process.

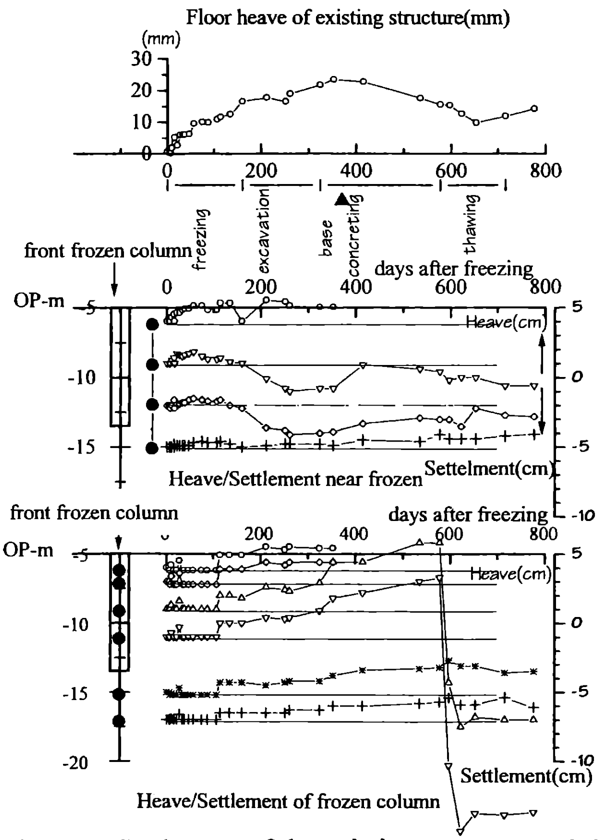

Figure 8. Settlement of the existing structure and the ground during freezing and thawing.

during the excavation stage and the base concrete slab was constructed.

3.5 *Thawing the frozen soil*

Since the soils had been extracted during freezing and excavation stages, settlement was expected not only by frozen process but also by the loss of soil volumes due to the extraction. To compensate the expected settlement, grouting into soft ground was performed as a positive counter measure. The grouting process is monitored to prevent any excessive heaving of the existing and the completed structures.

Interesting soil movement beneath the frozen column was observed. During the process of thawing frozen column A-line, hot water was circulated to accelerate the thawing the soil.

This horizontal movement was questioned if the sliding was taking place towards to excavated area. The displacement or heaving of the existing structure did not show any associated movement.

The displacement of newly built structures in the excavated area also did not show any meaningful change. The underground movement was estimated caused by softening within locally limited zone. Horizontal movement of the soil beneath the frozen front column became increased and finally reached about 15cm.

Based upon monitoring of heaving, extraction of soils were performed during freezing and subsequent excavation steps. No creep movement was observed after thawing and the geotechnical steps were successfully completed.

4 CONCLUSIONS

Based upon the case study of the freezing soft ground to excavate and to under-support the existing structure based upon observational procedure, the following conclusions were obtained.

1) The behavior of soft soil during freezing is monitored by measuring horizontal displacement, horizontal earth stress, pore water pressures.
2) Measured earth pressure at one point was used to estimate time change of stress distribution caused by a single pipe freezing.
3) The estimated distribution of stress and its change with time corresponded reasonably with the change of pore water pressure.
4) Soft clayey soils is shown very easy to expand due to freezing resulting large ground heave.
5) Observational procedure is shown as the good tool to control the geotechnical construction problems caused by ground freezing.

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