General Report A for Theme Nine – Case Histories in Subsidence of Soils, Tunnels and Shafts in Soft Ground, Waste Disposal Sites and Pavements

Hal Aldrich
Haley & Aldrich, Inc., Cambridge, Massachusetts

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Chairman of the Board, Haley & Aldrich, Inc., Cambridge, Massachusetts, U.S.A.

Forty-five papers were submitted for Session 9, Case Histories of Other Structures. Mr. Mansur and I have divided the papers, not in accordance with program titles for our two sessions, but rather to reflect our respective interests and experience.

CATEGORY I - Tunnels and Shafts
Papers 916,938,955,960 and 966
CATEGORY II - Slurry Trench Walls
Papers 925,934 and 943
CATEGORY III - Preloading
Papers 906,914,926 and 968
CATEGORY IV - Dynamic Compaction
Papers 908,920,941 and 967
CATEGORY V - Vertical Drains
Papers 912,915
CATEGORY VI - Miscellaneous
Papers 913,921,942 and 962

CATEGORY I - TUNNELS AND SHAFTS

Several good papers have been submitted to the conference summarizing detailed observations of ground movements during tunneling. With new and improved field instrumentation, accurate measurements become possible and greatly improve our understanding of in-situ soil behavior. A major sunken tube tunnel and a tunnel failure are also described.

Sarkar and Munfakh (No. 916) describe the geotechnical engineering aspects of the design and construction of the one billion dollar, eight-lane, 5,400-ft. long Fort McHenry tunnel across the harbor in Baltimore, Maryland. The immensity of this project and size of the binocular section immersed tube tunnel are the novel features of this paper, along with the required measures to accomplish an environmentally acceptable solution to dispose of dredged material.

The paper outlines the development of design drained and undrained strength parameters for the variety of subsoils present at the site, leading to recommendations for excavation slopes.

Excavation at the east end of the project required construction of a tied-back soldier pile and lagging lateral support system. A large number of tiebacks failed during proof testing by the contractor, in part due to inadequate anchor bond length. Nearly 100 production tiebacks were installed prior to the time results of 11 tieback anchor tests and design bond lengths were submitted to the authors for review. As a result, an extensive retesting and load-reduction program was necessary. Where possible, designers should monitor the performance of proof tests or insist on early submission of test results prior to installation of production tiebacks.

The authors are urged to publish details concerning the sequence, methods and technology for constructing the sunken tube tunnel.

Edgers, Thompson, Mooney and Young (No. 938) report on ground movements which occurred around twin 20-ft. diameter tunnels constructed for the MBTA Red Line Extension transit project in Cambridge, Massachusetts. The Test Section included three intensely instrumented sections in areas of rock, soft ground (glacial till) and mixed-face tunneling conditions approximately 100 ft. below ground surface. In addition to surface settlement monitoring, deep settlement points and inclinometer casings were installed to measure vertical and horizontal movements above and around the tunnels during and following advance of each heading.

Although the measured ground movements were very small, the results at the soft ground section add significant data points which agree with published relationships between tunnel depth/size and the settlement trough. However, the width of the settlement trough was wider, which the authors attribute to elastic rather than plastic deformations.

Data from the deep settlement points and inclinometers showed that large losses of soft ground at the tunnel heading propagated no more than one or two tunnel diameters from the heading. The authors suggest that this may be due to the ability of the dense glacial till to arch over any opening caused by ground loss.

The tendency for the dense predominantly granular soil to expand during shear displacement may counteract ground loss and result in less surface settlement volume, as suggested by Engels and Calabrese (No. 966) and others.

Literature on ground movements associated with tunneling under mixed face conditions is limited in comparison to soft ground tunnels in clay. This paper will be of interest for this reason as well.

The post-construction failures of three unreinforced concrete secondary-lined tunnels constructed through inorganic silts and fine sand near Detroit, Michigan are described by Neyer (No. 955). The 8.5 to 12.75-ft. diameter sewer or storm water tunnels were constructed from 20 to 30 ft. below the groundwater table. This paper provides important lessons for practicing soft ground tunnel designers, an opportunity to learn from the mistakes of others.

Piping of soils surrounding the tunnel through existing cracks in the secondary lining (primarily at cold and construction joints) or into an adjacent shaft under construction have led to loss of ground support around the tunnel lining, structural deformation and distress of the lining, and sometimes to total secondary lining collapse. The use of waterstops at construction joints, low cement factor concrete (to minimize shrinkage), pour lengths restricted to 120 ft. between construction joints are recommended design measures to addresss soil piping concerns in soft ground tunnels in similar geologic settings.

The appropriateness of unreinforced concrete tunnel linings constructed in similar subsurface conditions should be examined, even incorporating the author's design recommendations. Since concrete will shrink circumferentially as well as longitudinal following placement, longitudinal cracks at the tunnel springlines are as common as cracks at construction joints. If similar soils can pipe through cracks as small as 0.008 inches, as reported by the author, it is possible that piping through longitudinal springline cracks will occur even though the author's other design recommendations have been carried out.

Rather than conduct remedial grouting of all leaking longitudinal cracks upon the recommended visual post-construction inspection of the lining, light reinforcement, such as welded wire fabric, could be used to distribute total shrinkage crack widths into many narrower cracks around the tunnel lining circumference, each having widths smaller than the 0.008 inches. The issue of crack control in concrete tunnel linings could be expanded further by the author.

Lo, Karunaratne and Lee (No. 960) describe the results of geotechnical instrumentation for monitoring the ground response to construction of a timber-lined 2.1 m square sewer tunnel at a depth of 6.3 m through soft, highly compressible marine clays in Singapore. Results confirm relationships proposed by Peck (1969) and Yoshikoshi et. al. (1978) for curve fitting of surface settlements resulting from soft ground tunnel construction.

The authors conclude that when Lo's (1982) determination of standard deviation of ground loss and overload factor are included with these confirmed relationships, it should be possible, in principal, to predict the entire settlement trough for various tunnel diameters and depths in Singapore marine clays.

Engels and Calabrese (No. 966) report measured ground movements associated with the construction of 20-ft. diameter twin bore rapid transit tunnels for the Lexington Market Line in Baltimore, Maryland. Tunnels were constructed in both free air and compressed air through sedimentary and residual soils from 10 to 25 ft. below the groundwater table.

Ground movements are analyzed and compared with published data which the authors have summarized in Table II. The authors conclude that volume expansion of dense granular soils over the tunnels counteracted other sources of ground loss resulting in surface settlements smaller than those expected for similar tunnels reported in the literature. They also conclude that "the lowest percentages of ground loss apparently occurred as a result of shield overexcavation." The data appear to indicate that the greatest percentages occurred.

Segmented steel and segmented precast concrete liners were used in the inbound and outbound tunnels, respectively. It would be useful to have the authors comments on the impact of the two alternatives on resulting ground movements. It may be that the reduced width of the concrete liner segments (30 in.) compared to the steel liner segments (48 in.), and difference in construction procedure were effective in reducing the total volume loss. Was the same shield used for both tunnels and was the outside diameter of the steel and precast concrete linings the same?

CATEGORY II - SLURRY TRENCH WALLS

Applications for the use of concrete diaphragm walls constructed by the slurry trench method continue to expand and to be reported in the literature. Three papers to this conference are good examples of the diversity of this construction technique.

Zeigler, Wirth and Miller (No. 925) describe the design, construction, instrumentation and monitoring of concrete diaphragm walls constructed by the slurry trench method to provide lateral support immediately adjacent to high rise buildings during construction of the Charles Center Station for the Baltimore Metro in Baltimore, Maryland. Excavation by cut-and-cover methods using internal bracing extended 66 ft. below ground surface, well below spread footings which support the buildings.

The original contract specified the use of steel pipe piles, jacked to bedrock, to underpin existing foundations prior to construction of slurry walls. Construction difficulties and delays prompted changes in the design. The underpinning piles were eliminated and the slurry wall was stiffened as required to limit foundation settlement to acceptable values.

This paper demonstrates again the applicability and wonder of the slurry trench method. Soil conditions at this site were ideal with very compact silty sand and gravel having a Standard Penetration Resistance (N-value) of over 100 blows. Prior to slurry trench construction, soils for 10 to 15 ft. below spread footings of several buildings were chemically grouted to minimize loss of ground.

A field instrumentation and monitoring program for measuring wall deformation, strut loads, surface and building settlements and groundwater observation wells is well documented and presented.

Total settlement of seven adjacent major buildings ranged from approximately 0.3 in. to 0.8 in., with one exception at 1.0 in. No visual architectural or structural distress occurred in six of the seven buildings.

Moh and Song (No. 934) present two good case histories of the design, construction and monitoring of concrete diaphragm walls constructed by the slurry trench method in soft sedimentary deposits in Taipei City, Taiwan. Predictions of wall performance based on simplified theory and empirical relationships, when compared with the actual performance, suggest that an elastic model for the soil-wall system can reasonably predict the behavior of the diaphragm wall. Herein lies the authors' principal contribution.

The deflection of the wall in the two cases, Case A in a silty sandy soil where the wall bears in a compact sand stratum and Case B in a soft silty clay stratum without penetration to a hard stratum, is compared. The dramatic difference in behavior summarized in Figures 5 and 7 and in the "Conclusions" will also be influenced by the strut levels (especially the first) and prestressing which differ in the two cases. Nevertheless, the lateral movements in the wall which can occur well below the lowest level of excavation, approximately 3 in. in Case B, is clearly illustrated.

In the determination of external lateral loads on the diaphragm wall for calculation of strut loads, Figure 10, it is not clear to the General Reporter how hydrostatic water pressure was included.

Heine, McDonough and Singh (No. 943) describe the use of reinforced concrete diaphragm walls constructed by the slurry trench method to support 40-ft. high embankments and bridge structures for a depressed section of highway at the Williams Street Interchange in Atlanta, Georgia. The slurry wall was used in lieu of a conventional cantilever retaining wall. The advantages for this application were significant. One level of struts was constructed about 17 ft. above roadway grade while the base of the wall was supported by passive resistance in compact sandy silt soils below pavement grade.

Instrumentation and monitoring of wall movements is not described but actual movements "were in agreement with computed values" of 0.5 in. Details of computations made to establish the Contract limits of movement are not included.

An alternative to carrying the slurry wall to significant depths below pavement grade, encountering "rock-like" material difficult to excavate and causing delays, would be to incorporate a second level of struts just below pavement grade. The Authors' comment on this alternative is invited.

CATEGORY III - PRELOADING

Preloading, with or without vertical drains, has become a common method for soil improvement to reduce post construction settlement and to improve shear strength. Four case histories add to our knowledge of this technique.

Stamatopoulos and Kotzias (No. 906) present a case history describing preloading of a variable 35-ft. deposit of soft clays and loose sands prior to construction of an ore storage building located northwest of Athens, Greece. A long metal building having a roof supported by 3-hinged steel frames, was then founded on continuous reinforced concrete footings below the long walls. Had preloading not been used, the authors report that 50-ft. long piles would be required for building foundations and piles or stone columns would be needed under the floor.

A preload embankment 40 ft. high was constructed over a period of 250 days, to allow for consolidation and gain in strength required for foundation stability under the preload. After about 50 days under full load, primary consolidation was essentially complete and the compressible stratum had settled about 2.5 ft. (7%). In fact, piezometric levels in the deposit never exceeded about 7 ft. above their static levels. The project was well instrumented and monitored.

The authors provide great detail on subsurface conditions and soil properties both before and after preloading, documenting the changes in standard penetration resistance, water content, in-situ permeability and shear strength.

The criteria for maximum differential settlement between adjacent frames of 0.02 m is provided. However, no actual measurements on the completed operating structure are reported except that the "operation of all structural and mechanical elements has been satisfactory." We will await a report on actual total and differential settlements in a subsequent paper.

Youssef and Erol (No. 914) describe preloading for a large residential and hospital complex in Saudi Arabia. An average of 4 m of earth fill was used to densify a 10 m stratum of saturated loose silty sand underlain by 10 m of very loose sandy silt, resulting in from 20 to 30 cm of settlement within 30 to 40 days after loading. Fabric drains 2.5 m on center were used to accelerate the densification. The paper presents the time-settlement behavior during preloading and compares the results with predictions using various methods, including the Buisman-DeBeers method based on cone penetration resistances. The Buisman-DeBeers method overpredicted the magnitude of settlement by more than 50 percent.

The static cone penetration resistance was improved substantially by preloading to a depth of about 12 m but apparently not below that depth (in the very loose sandy silt). It is probable that some portion of the improvement, greatest near ground surface, occurred as a result of densification during placement of the compacted preload fill.

The authors do not report the extent to which the preload fill was removed following stabilization, if at all, nor the performance of the buildings constructed on the densified ground. The General Reporter questions the need for the fabric drains. A test section without the drains would have provided interesting data to compare with results reported in the paper.

Abbs, Cognon and Kelsey (No. 926) present the results of a ground improvement program, utilizing dynamic compaction, preloading and vertical drains, for devel opment of a large tank farm for a refin ery located in East Kalimantan, Indonesia. Soil conditions were highly variable from medium compact sands to soft marine clays commonly 30 m. deep. With out ground improvement, the edge settlement of tanks located in areas of deep clay was estimated to be as great as 1.2m. Furthermore, factors of safety against foundation failure were inadequate for in-situ shear strengths.

Precompression in areas of deep clay was accomplished with earth fills up to 11 m in height for 8 months maximum. PVC pipe wrapped with fabric was used for 50 mm drains spaced 1.2 to 2.0 m apart. Settlements of more than 2.5 m occurred in 8 months, a portion of which, the authors believe, was due to "lateral yielding of the soft clay".

Exploration and testing prior to construction were extensive and the contract work was carefully instrumented and monitored. Soil improvement was determined with vane shear, pressuremeters, and Dutch cone penetration tests.

During water testing, all tanks were monitored for settlement and the authors report the results for several representative tanks. Tank D-20-11, constructed over deep preloaded clay, settled approximately 0.2 m at the center and 0.1 m at the edge in 25 days.

This paper illustrates how various ground improvement techniques can effectively and economically be used where subsurface soil conditions are variable.

Lakner, Lee, Armstrong and Ingles (No. 968) describe the precompression, with PVC vertical drains, of a normally consolidated silty clay stratum having a depth up to 15 m to pretreat the ground prior to construction of one-story residential buildings located south of Sydney, Australia.

From settlement observations on a fill placed six years prior to geotechnical studies for the project, the authors calculated primary and secondary consolidation parameters. These values compared favorably with results of laboratory tests. Analyses to establish representative soil properties are presented in some detail in the paper.

PVC drains, 10 cm wide by 0.16 cm thick, were installed in pairs at 1.8 m spacing for a total of 20,178 drains. The paper provides actual costs for the pretreatment, and details and production rates for installing the PVC drains, information of value which is frequently missing from other papers. However, the details of the preload fill height, geometry and duration as well as observed settlements are not included.

The authors concluded that pretreatment by preloading reduced post construction settlements by a factor of seven, to values for the deepest clay of approximately 8 cm for both primary and secondary consolidation, the latter contributing 5 cm.

CATEGORY IV - DYNAMIC COMPACTION

The four papers submitted in this category illustrate the diversity of applications for dynamic compaction, introduced by Menard in 1970, for soil improvement, not only in material type but also geographically. The reader will note the following:

- No. 908 Municipal waste fill in Illinois USA, improved to support a one-story warehouse on shallow foundations.
- No. 920 Natural cohesionless soils and fill materials in Bangladesh and Spain, for support of large industrial plants.
- No. 941 Loose rockfill in British Columbia, Canada, improved to support a sawmill on shallow foundations, and
- No. 967 Natural sandy and clayey soils in China, full-scale instrumented tests.

The depth of soil improvement, D in meters, in the three cases reported, agreed closely with an empirical equation proposed by Leonards et al (1980): $D \approx 1/2 \sqrt{WH}$ where W is the weight in metric tons and H is the height of drop in meters.

Steinberg and Lukas (No. 908) present a case history of dynamic compaction to densify 50 ft. of municipal waste in Skokie, Illinois to prepare the foundation for a one-story steel-frame warehouse supported by spread footings. Slab-on-grade construction was used for live loads of from 400 to 500 lb. per sq. ft. The results of subsurface explorations, largely SPT values and pressuremeter modulus before and after dynamic compaction are discussed, as are the depths of soil improvement, offsite vibrations and the performance of the building foundations. The importance of field monitoring by qualified personnel is stressed.

The waste landfill, placed in an abandoned clay pit, had not been used since 1950. In addition, from 5 to 15 ft. of refuse fill were removed prior to the site improvement program.

Following an evaluation of test pounding, a 15-ton weight was selected to be dropped from a height of 60 ft. on 8-ft. centers for from three to five passes. Crushed rock was used to fill the craters and bring the site to subgrade level for the floor slab. The depth of improvement was approximately $0.58 \sqrt{WH}$, according to the authors. From 3.5 to 4.0 ft of compression occurred, approximately 12 to 13 percent of the 30-ft. waste fill thickness.

Building settlements from the time the footings were constructed to completion of the superstructure 6 months later were less than 2 in. The performance of the structure over the next 10 to 20 years remains to be determined.

O'Brien and Gupton (No. 920) present two well-documented case histories of the use of dynamic compaction to densify sites in Bangladesh and Spain for major industrial plants underlain primarily by finegrained normally consolidated silty sands with lenses and layers of silts and clays. At the Bangladesh site, 5 to 8 m of fine sand hydraulic fill occurred over the natural soils. Site improvement by dynamic compaction allowed the use of shallow foundations for proposed structures.

Field control methods included measurements of crater depth, induced settlement, SPT values, cone penetration resistance, pressuremeter modulus and pore water pressure.

The depth of effective compaction agreed reasonably well with Leonards (1980). In both projects, the soil column within that depth was shortened about 4 percent. The authors conclude "that cohesive soils having a plasticity index greater than about 15 cannot be improved by dynamic compaction without the use of supporting techniques as preloads and drains."

The performance of foundations for structures at the two sites is not included in the paper.

Wightman and Beaton (No. 941) describe the use of dynamic compaction to density up to 15 m of loose rock fill prior to construction of a sawmill, on shallow foundations, located in British Colombia. The rockfill, a coarse to fine grained sandstone, was reduced in thickness by an average of 5.6 percent.

The effectiveness of the compaction effort was determined by pressuremeter testing and by geophysical testing to measure shear and compression wave velocities, neither of which are ideal for quality assessment for rockfills. The authors describe in detail the difficulties encountered and interpretation of results.

Although no multiple surveys were performed to determine actual settlement of footing foundations, the authors conclude that no significant differential settlement has occurred following construction. This conclusion was reached by comparing surveyed base plate elevations 3 years after construction with design grades.

Densification of rock fills by dynamic compaction to reduce post construction settlement, seems to the General Reporter to be an ideal application of the method.

Wang and Deng (No. 967) describe several full-scale instrumented tests conducted south of Beijing, China to evaluate dynamic compaction for various soil profiles varying from fine sands to sandy clays. The purpose of the test program was to investigate the physical mechanism of soil behavior during impact compaction, evaluate the depth of improvement by monitoring soil vibrational response and establish standards for protection of adjacent structures during dynamic compaction.

The paper presents a mathematical approach for estimating and evaluating the effectiveness of dynamic compaction. The authors studied the physical behavior of soil particles during impact compaction by utilizing high speed photographic techniques and the measurement of shear compression and Raleigh waves vs. various depths and distances from impact points.

Soil deformation curves are presented and related to both cohesionless and cohesive soil behavior.

The calculated value for the depth of soil improvement from a complex formula developed in the paper is 6.5 m. The simple empirical formula proposed by Leonards et al (1980) for WH equal to 200 ton meters yields 7.1 m.

Criteria are presented for the determination of the effective number of blows required to compact cohesive soils based on measured pore pressure data. In addition, safe distances from an existing structure are presented based on vibrational analysis and measurements in both cohesive and cohesionless soils.

The results and conclusions of this paper add to the understanding of the physical mechanism involved in dynamic compaction.

CATEGORY V - VERTICAL DRAINS

In addition to the two papers included in this category, two of the four papers in Category II - Preloading describe case histories utilizing drains. Furthermore, several papers in other sessions include sand drains.

Aziz (No. 912) describes a new and unique vertical drain. a split-bamboo pole filled with coconut coir wicks, and its use with preloading in a test section located in Singapore.

The 9 m long bamboo drains, from 8 to 10 cm in diameter, were installed at 2.5 m spacing in a 50 by 100 m test section. The soil profile consisted of soft to medium sandy clayey silt and soft silty clay. An earthfill surcharge 2.5 m in depth was used for a period of 2 years. Settlement and porewater pressures were observed during consolidation.

Soil properties before and after consolidation, in particular the natural water content, SPT values and unconfined compressive strength are reported and discussed briefly. Results of porewater pressure measurements from piezometers are not discussed.

The author states that the bamboo drains have positive advantages over other types of vertical drains, but those advantages are not identified. The General Reporter believes that the use of bamboo drains will be limited to cases where bamboo and labor are plentiful and inexpensive, where the depth of soft soil is less than about 10 m, and where considerable time is available to achieve stabilization.

Long and Hover (No. 915) describe the performance of jetted 12-in. diameter sand drains beneath a preload fill to consolidate about 60 ft. of stiff to soft marine silty clay for a wastewater treatment plan in Gloucester, Massachusetts. Data from field instrumentation in this well documented case history are summarized and used to compare observed and predicted behavior. Back-figured soil parameters are compared to laboratory values. Preload duration was long enough to observe secondary compression.

Statistical methods using computer analyses were used to backfigure primary settlement, and coefficients of consolidation and secondary compression. This could be a useful procedure to compare field observations with predictions based on laboratory data.

Field results compared favorably with predictions based on results of laboratory tests.

A discussion of the possible influence of vertical drain well resistance would have been useful. Well resistance may have been a factor in retarding consolidation since 1) drain lengths were great in some areas, up to 90 ft., and 2) organic soils above the clay probably discharged relatively large amounts of water. This could also have affected the backfigured values of c_h.

The performance of the structures constructed after site stabilization is not included in the paper.

CATEGORY VI - MISCELLANEOUS

In this last category, there are four papers concerning mining subsidence, the use of micro-piles and a case history of a house foundation on seasonally frozen ground.

Karmis, Triplett, Schilizzi and Hasenfus (No. 913) compile data from a series of cases of mine subsidence in the Appalachian area in the eastern U.S. Recommended standards for ground deformation measurements in subsidence areas are proposed. The authors are currently testing the standards "in a systematic monitoring program being pursued above four mines in southwest Virginia. In addition, some basic subsidence relationships are described in detail for both longwall and room and pillar panels, with particular emphasis on predictive capabilities. Finally, the application of these relationships is discussed in terms of improved engineering design."

The authors have attempted to account for the influence of geology in subsidence prediction. This influence is not typically accounted for in empirical approaches.

This paper is an important step in developing an empirical system for subsidence prediction in the eastern portion of the U.S. There is a need for this study since previous empirical approaches are based on data from Great Britain, Russia, Germany and Poland. The geology in the U.S. is different from these areas and impacts subsidence models. Also, there is a need for establishing uniform guidelines for monitoring subsidence as the authors have done.

Siriwardane and Moulton (No. 962) present a case history of a landslide, which occurred over an undermined area, and which caused major damage to a building at the toe of the hillside located in West Virginia. The investigation included a finite element analysis and concluded that subsidence from mining activity was a direct cause of the structural damage. Major effects included changes in the groundwater flow pattern and development of large tensile cracks in the hillside behind the structure, which triggered a block type slope failure in a "fire clay" layer.

Field data are lacking in this case history to substantiate the authors' hypothesis. Instrumentation to monitor horizontal deformations and pore pressures in the hillside before and after tension crack formation would have been instructive.

Singh and Heine (No. 942) describe in detail the use of pressure-injected micro-piles to upgrade spread footing foundations supporting an existing two-story rigid frame steel building located in Tyler, Texas. Micro-piles were installed through existing footings to reduce anticipated settlement to tolerable limits when a new mezzanine floor was added. Footings were bearing on loose silty fine sand.

The authors describe the design and installation of the micro-piles and theoretical load capacity and estimated settlement. One compression load test to 60 tons was performed, resulting in a final settlement of 0.43 in.

Micro-piles were 1.25-in. Dywidag bars grouted into a 6-in. diameter hollow stem augered hole. Neat Portland cement grout was used under low pressure. The pressure grouted zone was approximately 25 ft. long, calculated to provide a factor of safety of 2.0.

The installation was reported to be successful and effective although no settlement data for the foundations when the mezzanine loads were added is provided.

Xu and Wang (No. 921) describe the results of field measurements below and adjacent to a masonry building in China, a building having a shallow foundation bearing on clayey soil and subject to frost heave during winter months. The depth of foundation was 0.7 m while the maximum depth of frost penetration was approximately 2.0 m. Ground temperatures, depths of frost penetration, contact pressures at the bottom of the foundation and frost heave were all measured at numerous locations during seasonal freezing and thawing. The pattern of cracks in the building was also observed.

As expected, there were significant variations in the measurements depending on their location, whether at the corner or along the side of the heated building, whether on the north or south side and whether below the inside or outside edge of the footing. These variations led to differential heaving and cracking of exterior walls and foundations.

While only representative data are reported, the relationships appear to be reasonable. In a subsequent paper, the authors are encouraged to report all the data including the seasonal variation in air temperature, the building temperature and foundation cross-sections. A detailed description of the instrumentation would also be of interest. The authors do not mention the possible presence of snow which would have a significant effect on ground temperatures.

The authors developed a frost heave classification for clayey soils based on the water content in relation to the plastic limit. They conclude that there is an allowable thickness of frozen soil below the foundation and they provide a formula for calculating the minimum depth of foundation which is less than the maximum depth of frost penetration for the region.

SUMMARY

In summary, the papers presented to this session on tunnels, slurry trench walls, soil improvement techniques and miscellaneous topics add significantly to the record of case histories in geotechnical engineering.