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General Report on Session No. 5: Case Histories of Retaining Structures and Deep Excavations

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Case Histories of Retaining Structures and Deep Excavations

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INTRODUCTION

This session contains twenty-nine papers submitted from thirteen countries. The largest contingent - thirteen papers - is from the United States of America. The papers deal with a broad range of subjects within the over-all topic of full-scale case histories of retaining structures and deep excavations.

In order to provide some structure for the General Report and to apportion the related tasks of preparing a report and discussion on the papers, the General Reporter and Co-Reporters divided the papers into six categories. It must be understood that the categories are neither rigid nor exclusive; many, perhaps most, papers could easily fall into more than one category. However, the categorization does give a framework for reporting and discussion. The categories into which the papers are divided for purposes of this report are:

- Failures

Each of the six papers in this category describes and evaluates either the failure of a retaining wall or deep excavation or the failure of a number of related structures. The principal reporting was done by C. Soydemir.

- Project Descriptions

The nine papers in this category describe the design or construction of a retaining wall or deep excavation. The major emphasis is on describing what was done rather than on analysis, prediction, or comparison of field measurements with theory. The principal reporting was done by J. T. Christian.

- Comparison of Field Performance and Prediction

The seven papers in this category are concerned with comparing field observations with analytical studies, whether made before, during, or after the fact. The principal reporting was done by A. J. Lutenegger.

- Analytically Controlled and Instrumented Construction

The three papers in this category describe projects in which analytical studies and instrumental observations were used to control, revise, and modify the construction in progress. The principal reporting was done by C. Soydemir.

- Analytical or Model Studies

These two papers describe analytical or model studies of issues arising in retaining wall design. The principal reporting was done by J. T. Christian.

- Flow through Porous Media

Two papers deal with groundwater flow through porous media and its effect on the behavior of excavations. The principal reporting was done by A. J. Lutenegger.

FAILURES

Five of the papers submitted to the conference in this category are brief case studies of failures or unacceptable performance of earth retaining structures. Four of these deal with temporary excavation support systems, and one reports on a permanent retaining wall project. The sixth paper presents a statistical evaluation of fatal accidents caused by the failure of trench excavations in Japan.

Temporary Excavation Support Systems

Rodgers and Majchrzak (paper 5.15) describe the unacceptable performance of a steel sheet pile wall supported internally by a berm and raker system. During the course of a 8.5 m (28 ft) deep excavation in downtown San Francisco, California, the lateral support system moved more than 450 mm (18 in) horizontally, resulting in the loss of usable basement space and serious distress to adjacent streets and utilities.

The case is an example of poor communication between the geotechnical engineer and the contractor, who also happened to be the owner. It is especially relevant to note that the loss of ground due to the removal of existing wood piles from the area of excavation was a major contributor to the ground movements. Contrary to the engineer's recommendation, 300 piles were removed within the zone of influence of the support system, and the resulting voids were not backfilled sand or grout.

Rahimi et al. (paper 5.23) report on the unacceptable performance of two sewerage mains constructed across two creeks using a temporary embankment and a sheet-piled trench excavation.

Following the construction of two effluent mains in very soft deposits in central New South Wales, Australia, joint openings up to 120 mm (4.75 in) wide developed. The pipes were 600 mm (2 ft) and 750 mm (2.5 ft) in diameter.

This case is an example of the importance of construction induced displacements during surcharging, excavation, backfilling, and removal of surcharge in very soft deposits, which should be estimated within a reasonably confined range to determine the technical feasibility of a particular construction scheme proposed by a contractor. The paper describes analytical efforts to simulate the construction process in order to understand the mechanisms that led to the unsatisfactory performance.

Olson and Heuer (paper 5.32) describe the failure of a 36 m (117 ft) diameter circular excavation support system consisting of steel ring beams and vertical timber lagging. The braced excavation for a new sewage treatment plant in central Texas failed as the excavation reached a depth of about 13 m (42 ft). The total design depth of the excavation was 27 m (88 ft). The support system was designed by a well-known international engineering firm based on subsurface information obtained from a single test boring made at the center of the excavation during the dry summer season for the region.

An unanticipated water-bearing sand and gravel layer was encountered within one segment of the excavation, which the single test boring did not find. As the excavation proceeded during the rainy season, there was progressive loss of material into the excavation. This could not be controlled and led to the buckling of the ring beams and total collapse of the support system.

The case is a well-documented example of deficiencies and errors contributed by the contractor and the designer, as well as the owner.

Horii et al. (paper 5.39) report on the catastrophic failure of a soldier pile and timber lagging support system stabilized by soil anchors. As the excavation at a site in Japan reached a depth of 11.4 m (37 ft), several soldier piles (steel H-sections) braced by a single level of wales and soil anchors became unstable due to the loss of toe support. Five workers were killed under the sliding earth mass.

Apparently, the failure was caused largely by the gross error of the field engineer, who changed the original design for the excavation support system to obtain "cost savings." Also, the soldier piles were installed short of their required tip elevations because of a mistake in the base datum for the project. In addition, the design was based on subsurface information obtained from a single test boring and did not consider any hydrostatic pressure build up, whereas after the failure it was established that heavy rainfall did cause hydrostatic pressures on the support system.

Permanent Retaining Support Systems

Olson (paper 5.31) describes the failure of an earth retaining system consisting of two adjacent retaining walls. A composite earth retaining system, consisting of two reinforced concrete cantilever walls, the lower 5.2 m (17 ft) high and the upper 3.4 m (11 ft) high, collapsed shortly after completion of backfilling at a site in central Texas. The system was designed by a registered professional engineer in accordance with a standard handbook. However, he did not consider the surcharge effect of the upper wall on the lower wall and the over-all sliding stability of the composite system.

The case is an example of a grossly deficient design analysis undertaken by an unqualified designer, who reportedly was a "generalist having performed design work in most areas of civil engineering but not trained in geotechnical engineering."

Statistical Evaluation of Fatal Accidents Related to Trench Excavations

Toyosawa et al. (paper 5.22), of the Institute of Industrial Safety, Japan, made a comprehensive survey of the fatal accidents, between 50 and 100 deaths each year, related to excavation of trenches to install utility pipes at relatively shallow depths in urban areas of Japan. The statistical evaluation reveals that nearly 80 per cent of the accidents occurred in trench excavations less than 3 m (10 ft) in depth and were due to the sliding of urban fill materials. At the times of the failures, about one half of the excavations did not yet have the support system installed, and about 30 per cent of the failures occurred during the installation or dismantling of the support system.

The authors and those interested in safety of trenches my find relevant information in the regulations of the U. S. Department of Labor, Occupational Safety and Health Administration (1989).

General Comments

The cases reported come form the United States of America, Japan, and Australia. Thus, they reveal some differences in the way the relationships between the owner, designer, and contractor work for excavation support projects in different countries. However, the elements that contributed most significantly to failure or unacceptable performance are quite similar.

In the four reported failures of temporary excavation support systems, lack of adequate information on subsurface conditions and the complete separation of the designer from the construction phase activities stand out as the primary causes leading to unsuccessful projects and unfortunate results.

Temporary excavation support systems are almost always designed by the contractor because of liability concerns. However, the project geotechnical engineer is the person most informed on the subsurface conditions underlying the project site, and he or she is therefore in the best position to assess the anticipated lateral earth pressures and ground movements associated with a particular excavation support system. In essence, the design and construction of an excavation support system is a soil-structure interaction problem, which requires a direct and continuous contact between the project geotechnical engineer and the contractor.

Finally, it is interesting to observe that, in each of the five cases of failure reported, during the ensuing post-failure period eminent consultants were retained, and substantial sums of money were expended for extensive subsurface explorations, thorough laboratory testing programs, and sophisticated analyses. Ironically, if some of these efforts had been undertaken even at a modest level during the design phase, the subsequent catastrophes and costly litigation might have been avoided.

PROJECT DESCRIPTIONS

Nine of the papers presented to the session are essentially descriptions of projects. Four of the papers deal with the performance of various types of retaining structures. Two other papers describe projects that used reinforced soil techniques, and two report on the underpinning of existing structures. One paper presents the case of a pressure relief tunnel.

Retaining Structures

Hohmeyer (paper 5.08) describes the design and construction of two retaining walls for an addition to a hospital in Michigan. One wall was a temporary structure, and the other was permanent. Both were constructed as augercast piles; that is, the hole for the pile was drilled with an auger and then filled with Portland cement grout pumped out the bottom of the auger. The scheme was chosen because the lower portion of the soil profile consists of very stiff clay in which adequate embedment might be difficult to obtain. Also, the augercast technique reduced the noise and vibration during construction.

The permanent wall was designed on the assumption that each pile acted independently. The computed factor of safety was 2.0. The temporary wall was designed as a set of piles forming semi-circular horizontal arches. The computed factor of safety was 1.5.

Both permanent and temporary walls performed well. However, during construction it was observed that the piles in the temporary wall were not in contact, as had been assumed in design, but were separated by about 50 mm (2 in) of clay in a "smear zone" that developed when the cuttings from the auger were forced into the sides of the hole. The shear strength of the clay was measured with a hand penetrometer to be at least 21 mPa (3000 psf), which provided sufficient interlock to maintain the arch.

Woo et al. (paper 5.18) present a very clear and complete description of the construction of a pier in Quincy, Massachusetts, under very difficult conditions. The project was to replace an existing pier in an area of miscellaneous fill, organic deposits, glacio-marine deposits, and till. Because of the presence of existing buildings, tanks, and other structures, three different construction schemes were employed along different sections of the pier. The longest portion was an anchored bulkhead made up of 18 m (60 ft) long steel sheet piles anchored to a concrete deadman by tie rods on 2 m (6.5 ft) centers. Next came a 24 m (80 ft) relieving platform consisting of vertical and battered concrete-filled steel pipe piles supporting a reinforced concrete deck. The last section was nearly perpendicular to another nearby pier, and the designers chose for this section a double wall of 21 m (70 ft) long sheet piles connected with anchor rods and backfilled with crushed stone.

Inclinometers were placed on all sections and monitored throughout the several phases of construction and into the later life of the structure. Movements were acceptably small. In the first two sections total maximum horizontal movements were approximately 50 mm (2 in) to 75 mm (3 in). Movement of the last section, consisting of the double sheet pile wall) exceeded 140 mm (5.5 in), but the rate of movement decreased with time. The performance of the new pier has been satisfactory.

Teparaska (paper 5.21) describes the behavior of the braced retaining system for an 11.3 m (33 ft) deep excavation for the basement of and 89 story building in Bangkok, Thailand. The site is underlain by very soft, thick deposits of marine clay with undrained shear strengths from 6 to 20 kPa (0.8 to 2.8 psi) to a depth of about 27 m (90 ft). The building is in two sections, one 19 stories high with an excavation depth of 9.5 m (31.4 ft) and the other 89 stories high with an excavation depth of 11.3 m (37.3 ft).

The bracing for the sheeted excavation consisted of three levels of struts at the shallow section and four rows at the deep section. Struts were preloaded to 70 per cent of the apparent pressure from the standard Terzaghi and Peck pressure diagram. Before each level of struts was installed the wall was partially supported by berms. Four inclinometers were installed, and their readings are reported for the various stages of construction.

The observed motions of the walls involved rotation about the bottom and bulging in the middle, primarily below the level of excavation. This is what would be expected for this type of construction. Traffic beside the wall had a significant effect on the lateral motions and settlements; indeed, the restriction of truck traffic seems to have reduced the horizontal movements during the initial stage of excavation as measured by inclinometers at two locations from about 80 mm (3 in) to about 24 mm (1 in). Berms were effective in reducing movements only in the first three stages of excavation. The ratio of maximum horizontal wall movement to excavation depth as a function of factor of safety against basal heave was generally less than that reported by Mana and Clough (1981) for several sites around the world. A simplified method, proposed by Wong and Broms (1989) for predicting horizontal wall movement, gave results that agreed well with the field observations.

The paper presents much detail on the design and performance of the excavation and warrants further study. It is particularly relevant for excavations in deep deposits of very soft clay.

Matsui and Nakajima (paper 7.28) describe the field measurements on a small diaphragm wall constructed as part of the foundation works for an elevated highway near Kobe, Japan. The soil consisted of about 20 m (66 ft) of fill overlying alternatively bedded sands and clays. The wall was built inside a box created by other concrete walls, so that dissipation of excess pore pressures was reduced. Measurements were made of the pore pressures and accelerations in the soil. The data showed that pulses of acceleration were generated when the excavating bucket struck the soil during excavation. This generated excess pore pressures, which decayed much more rapidly outside the box created by the other walls than inside the box. The authors conclude that the excess pore pressures could reduce the stability of the wall and that a deep well provides an effective means of dissipating the excess pore pressures.

Reinforced Soil

Schick et al. (paper 5.13) report on the behavior of a Reinforced Earth® wall built to contain the sliding of earth near an office building in Houston, Texas. The foundation soils appear to be composed primarily of random, uncontrolled and unconsolidated fill. The instrumentation consisted of four inclinometers and seven sets of horizontal displacement markers, supplemented later by eight sets of horizontal displacement markers located in the area of greatest movement. Inconsistencies in the data led to the conclusion that the inclinometers had not penetrated to a point of fixity, and the inclinometer readings were discontinued.

The maximum horizontal movement of the wall has been about 610 mm (24 in), and the maximum settlement, which occurred at the same section of the wall, has been 660 mm (26 in). The monitoring data indicate continued creep movement in an area of a former slide. The progress of the movements has coincided with the times of heavier rainfall. Although the movements are large, the wall seems to have accommodated them, and the structure does not appear to be in danger. Several explanations of the ongoing movements are offered. Continued monitoring and further study is recommended.

Jamnongpipatkul et al. (paper 5.30) describe the design, construction, and performance of a reinforced soil wall along the alignment of a highway in northern Thailand. The profile consisted for the most part of residual soils produced by in situ weathering of diorite, underlain by weathered rock and intact rock. Design was based on empirical correlations between standard penetration test results and soil properties. The residual soil was replaced with structural backfill above the weathered rock.

As pore water pressures were considered a major problem, an extensive system of underdrains was installed. Settlement plates, Casagrande piezometers, and inclinometers were installed to permit monitoring the performance of a test section. This showed that inadequate compaction and heavy truck traffic could lead to local failure of the wall. No excess pore pressures were observed. However, horizontal movements in excess of 300 mm (12 in) are reported, as well as substantial differential settlements. Better attention to compaction details is expected t improve the performance of future structures.

Underpinning

Lim and Majchrzak (paper 5.16) report on th underpinning of a four-story building in Redwoo City, California. Geotechnical considerations a well as operational constraints due to th current use of the building as a jail led to th examination and rejection of most conventiona alternatives for underpinning the structure. Th site is blanketed by about 1.4 m (5 ft) of highl expansive black clay with a plasticity index o 44, underlain by variable mixtures of mediu stiff to stiff silty clay and sandy clays an medium dense to dense clayey sands. These soil are characteristic of the fluvial depositiona history at the site. Underpinning was require to keep the existing structure in place durin construction of a ten-story addition.

The designers selected a system of drille underpinning piers to support the existin building during construction. The piers wer designed to carry not only the vertical load along the perimeter of the building but als horizontal loads due to lateral earth pressur and the lateral pressures from existing interio footings. The resulting design consisted of 61 mm (24 in) diameter piers spaced 0.6 m (2 ft) o centers where the interior spread footings ha greatest influence and 2.4 m (8 ft) elsewhere The piers extended a minimum of 1.5 m (5 ft below the elevation of the bottom of th foundation for the new building. To provid lateral reinforcement and support, the pier included W12X53 H beams. The system wa restrained by tiebacks.

The designers specified specifi construction steps to minimize movement an disturbance during installation of th underpinning system. Seventeen monitoring point were installed to monitor movement of th building. Maximum vertical motion was 9 mm (0.3 in), which was within the acceptable range.

The authors observe that the system employe was cost effective, but the small movement encountered suggest that further cost reductio might have been possible. They state that thes measures would have been facilitated by selectio of the shoring engineer on the basis o competence and experience rather than competitiv bidding. Further, they recommend that th shoring engineer should be made a member of th design team from the beginning of the project.

Marangos (paper 5.36) describes underpinnin a four-story building near Kastoria Lake i Greece. The building had tilted, experiencin 167 mm (6.6 in) of differential settlement. Th main cause of the differential settlement seem to be the presence of old artificial fills alon one side of the building and organic soils on th other. Underpinning involved installing 21 bore piles to support the mat at the softer side o the site and a six-pile bearing wall placed nea the edge of the mat.

Pressure Relief Tunnel

Graham et al. (paper 5.26) describe th design and construction of a 590 m (1945 ft) lon tunnel behind a four-tier, anchored, tied-back retaining wall. The project was part of the stabilization of the slope for a highway interchange in Steubenville, Ohio.

Analysis of the stability of the retaining wall revealed that pore pressures in the bedded shales, sandstones, and coal seams behind the wall would lead to instability by block sliding unless measures were taken to relieve the fluid pressures. The solution was a pressure relief tunnel. The contractor selected an inverted U-shaped tunnel, 2.4 m (8 ft) wide by 2.7 m (9 ft) high. Because the tunnel is located at a relatively shallow depth, its stability is controlled by the structure of the rock rather than overstressing of the rock. Unstressed rock bolts and shotcrete provided adequate support for the rock in the tunnel. Piezometers were installed from the tunnel along its length to monitor the drawdown of the water level.

The paper describes details of the construction process. Piezometer readings since 1991 indicate that the tunnel has effectively drawn down the pore pressures and stabilized the retaining wall. The reduction in required tie-backs for the retaining wall has provided a net saving of seven million dollars.

General Comments

These papers describe a wide variety of projects. A consistent theme in those cases that could be considered engineering successes is the early involvement of geotechnical engineers and other professionals in the design decisions. The papers also demonstrate the great utility of intelligently designed field instrumentation to monitor the behavior of a retaining structure, especially in difficult soil conditions.

COMPARISON OF FIELD PERFORMANCE AND PREDICTION

Seven papers presented to this session deal with the observed behavior of geotechnical structures and its comparison to predictions. Five of these describe the behavior of retaining systems for excavations. They include performance of slurry walls, braced sheet pile walls, and reinforced concrete diaphragm walls. The two remaining papers report on the performance of underground structures, namely a long-span arch culvert and a soft-ground tunnel.

Walls

Tamaro et al. (paper 5.06) present the summary of the performance of a structural slurry wall for the Washington, D. C., Metro. They examine the use and accuracy of four methods of analysis for the wall: the use of the Terzaghi and Peck loading diagram, the net pressure method with support settlements, the beam on elastic foundation method, and the finite element program SOILSTRUCT. They describe the methods and the factors that influence the predictions of each. They conclude that, for the wall analyzed, the beam on elastic foundation method was most appropriate. They find that the results of finite element calculations depend strongly on the choice of values for soil properties and that, in this case, the technique greatly over-predicted the deformations and movements.

Edstam and Jendeby (paper 5.14) present results of a study to evaluate the earth pressure distribution and displacement of a braced sheet pile wall for a 6 m (20 ft) deep excavation in soft clay in Sweden. Instrumented sheet pile sections showed that the clay was normally consolidated with $K_0 = 0.7$ prior to excavation. These measurements are especially interesting in that they evaluate existing stress conditions. The prediction of earth pressures and wall movements was made using the finite element code FLAC. From the earth pressure measurements it was clear that FLAC did not simulate well the earth pressure distribution caused by the excavation and gave predictions higher than the measured data. The results also show that earth pressures can be very different from those predicted by Peck's diagram and emphasize that engineers should be aware that the diagram is intended as an envelope for purposes of conservative design and not as a tool for predicting field observations. The authors also found that the method of the beam on elastic foundation worked well when the parameters were varied to obtain the best fit. The field stresses were found to be relatively insensitive to soil properties, but the predicted displacements were very sensitive to the properties used.

Abedi et al. (paper 5.35) describe the modeling and behavior of a braced sheet pile wall for an excavation 7.2 m (24 ft) deep in soft clay in Detroit, Michigan. Three rows of sheet piles were used for adjacent excavation support. Analyses of the wall movements were made using the finite element program SOILSTRUCT. The observed horizontal movements, which were obtained primarily with inclinometers, did not correspond to predicted movements in all cases, and the authors attribute the discrepancies to several possible factors. These include: the three-dimensional nature of the actual project compared to the two-dimensional analytical model, errors in assumed soil properties, and deviations between the assumed construction sequence and that used in the modeling. The authors consider the last to be probably the most important and suggest that more refined predictions could be achieved by more detailed modeling of the construction sequence.

Lin and Deng (paper 5.40) report on the behavior of a reinforced concrete diaphragm wall used to support a deep excavation in Taipei, Taiwan. The paper is of interest in part because the excavation was very deep - on the order of 22 m (73 ft) - and was part of the construction of a 27 story building. The soils are a layered sequence of deposits consisting primarily of silty clays and silty sands. The authors used the results of instrumentation readings along with the finite element code SOILSTRUCT to update design and provide construction control. For different stages of the excavation, the authors found good agreement between the predicted and measured displacement of the wall, but this may be the result of updating the soil properties and using monitoring feedback in the analysis. In particular, the soil in the model was strengthened significantly following the observations on the first three stages on construction.

Moh and Hwang (paper 5.44) describe the observations on a similar project, in this case a 17 m (56 ft) deep excavation in Taipei, Taiwan. They report on the earth pressure measurements and wall movements for a diaphragm wall. The authors state that the simple use of overburden pressures is not correct for assessing stability and suggest further that wall friction amounting to $\delta' = \phi'$ should be used at "soft to medium stiff" sites for design of braced retaining structures using beam models. Passive pressure coefficients as high as 9 were recorded. As the authors rightly point out, earth pressures are functions of wall movement and limiting values will be developed only if sufficient wall movement takes place.

Underground Structures

Byrne et al. (paper 5.45) describe the construction and performance of a long-span arch metal culvert installed in British Columbia as a replacement for a collapsed structure of identical design. Ultimate cover over the crown of the new culvert consisted of 9.6 m (31.7 ft) of soil. The displacements at the crown and spring line and the vertical and horizontal earth pressure over the crown were compared with values calculated by the finite element code NLSSIP. The comparison showed that positive arching was occurring in the soil, reducing the soil stress above the crown and the axial stress in the arch. Finite element analyses suggest that low measured values of thrust were the result of slippage at bolted joints, which has been confirmed by recent laboratory tests.

Parreira and Azevedo (paper 5.55) present the results of predictions and measurements of displacements around a tunnel in soft ground in São Paulo, Brazil. The tunnel was constructed in a layered profile of soft and stiff clays with a soil cover of about 7.6 m (25 ft) above the tunnel's crown. Measurements of soil movements were made from surface settlement points and a slope indicator. Numerical simulation employed the finite element code ANLOG, which uses an elasto-plastic model for soil behavior and seems to be based on the SOILSTRUCT program. The material properties were verified by comparison with the results of consolidated drained triaxial compression tests on samples of the clay. The authors report good agreement between calculation and observation for surface movements at increasing distance form the centerline of the tunnel and horizontal movements adjacent to the tunnel.

General Comments

These seven papers describe comparisons between calculated and observed behavior for a variety of soils and construction conditions. In many cases it is clear that significant iteration is needed to get good agreement,and usually this iteration involves the soil properties. The selection of soil properties for full, before-the-fact (Class A) prediction remains a difficult and chancy operation. This is one reason that simpler, more empirical methods, such as the beam on elastic foundation, which require a smaller number of soil parameters and may be less sensitive to errors in the selection of poorly defined parameters are often found to be more useful in design. It is also not clear that the users fully understand the details of the finit element analyses.

ANALYTICALLY CONTROLLED AND INSTRUMENTE CONSTRUCTION

Three papers submitted to the conferenc describe case studies of analytically controlle and instrumented construction of excavatio support systems.

Summary of the Papers

Bray et al. (paper 5.25) present a cas study of an 18.3 m (60 ft) deep, 15.3 m (50 ft wide, and 190 m (625 ft) long braced excavatio for the removal and reconstruction of tw parallel, 4.3 m (14 ft) diameter intake/discharg pipelines on the southern shore of Lake Michigan The sudden collapse of the old corrugated stee pipes created a 26 m (100 ft) wide and 6 m (2 ft) deep sinkhole, partially undermining th foundation support for several critica structures at the site. Excavation fc reconstruction of the pipes required tha horizontal and vertical ground movements b limited to about 25 mm (1 in) to protect th integrity of the existing structures, which wer already distressed.

Extensive finite element analyses wer undertaken to determine the most technically an economically feasible excavation support systen PZ 40 steel sheet piling with tiebacks an supplementary internal bracing was selected. Th results of two-dimensional plane-strain finit element analyses were adjusted for the beneficia effect of three-dimensional behavior.

The performance of the excavation durin construction was monitored by a comprehensive se of instruments, including piezometers inclinometers, surface monuments, tilt meters electro-levels on the existing adjacer structures, strain gauges on the struts, and loa cells on the tiebacks. Performance dat collected during the initial, less critical phas of the excavation were employed to modify th finite element program in making predictions fc more critical, advanced stages of the excavatior

Lien et al. (paper 5.48) present a cas study of a 6.4 m (21 ft) wide, 8.5 m (28 ft) dee trench excavation in the soft clays of Detroit Michigan, supported by semi-rigid tangent wall that were cross-braced at five (5) levels b steel strut members. The tangent walls consiste of 105 mm (42 in) diameter drilled concrete pie with W36-230 steel core members. Because of th strict requirements on the permissible latera movements, a comprehensive finite elemen analysis was undertaken in the design, and a extensive instrumentation program was implemente during construction.

Performance of the walls monitored throuc the full excavation agreed well with th analytically estimated magnitudes. The maximu lateral movements of the walls were under 50 n (2 in).

Due to the time constraints for th preparation of the paper, the authors could nc include performance data obtained during th construction of the chamber structure within th buried excavation. Also, a relevant item is the interference of the five-level wales and struts, spaced at 1.5 m (5 ft) intervals, with the construction of the chamber box, and how this problem was handled while meeting the strict performance requirements.

Pöttler (paper 5.53) describes the design and construction of a 200 m (655 ft) long segment of a tunnel along the Hanover-Würzburg rail line in Germany. This tunnel was planned to be constructed in an approximately 30 m (100 ft) deep open excavation with sides sloping 60 degrees from the horizontal in relatively poor rock and soil overburden, which would then be backfilled. In other words, the designers contemplated a cut-and-cover operation.

Deformations of the tunnel section upon backfilling were estimated using the "beam element model," which takes into account the structural characteristics of the tunnel section, and the soil-structure interaction is represented by linear elastic springs. Different values of modulus of subgrade reaction were assigned for each of the tunnel's characteristic support zones. Early in the construction phase the tunnel geometry was modified slightly without additional analysis.

Roof and invert settlements, as well as horizontal divergences of the tunnel section, were monitored during backfilling of the first 9 m (30 ft) section above the roof of the tunnel. At this juncture the measured deformations reached levels that were twice the estimated magnitudes, and cracking in the roof and invert of the tunnel was observed. The backfilling was halted and an extensive finite element analysis of the modified design section was undertaken to confirm the safety of the structure. The analysis established that the deformations of the tunnel were governed almost exclusively by the moduli of subgrade reaction. Back-calculation also established that the modulus of subgrade reaction in the transition zone from he invert to the bench area was about one-tenth of what was used in the original design. Finite element analyses with the more representative values of modulus disclosed that the stability of the tunnel would be maintained during the remaining portion of the construction if less thickness of backfill were placed than originally planned. The new scheme was implemented, and the tunnel has been in operation since 1987 without any problems.

General Comments

The three case studies reported are successful examples of using state-of-the-art analytical procedures in conjunction with construction performance monitoring to deal with critical excavations and backfilling projects. Where ground displacements are to be controlled within only a few centimeters to maintain the operation and to protect the structural integrity of existing critical structures, the design and construction of excavation support systems become challenging tasks for the designer, the contractor, and the owner. The success of such delicate undertakings depends primarily on the close and positive interaction among these three parties.

Use of construction monitoring in conjunction with analytical procedures, and especially with finite element procedures, offers wide capabilities in the design and construction of complex excavation support systems, which could not have been undertaken earlier. Such an approach also enables the engineer to determine quite intricate and hard-to-obtain soil and rock properties and soil-structure-interaction parameters by back-calculations, which can then be used in further analyses.

ANALYTICAL AND MODEL STUDIES

Two papers presented in this session deal with analytical and model studies, one in each category. The studies are both motivated by design problems arising in practice.

Saran and Khan (paper 5.03) describe tests conducted on a 4 m (13.2 ft) high model of a reinforced soil wall. The soil was a uniform sand, and three reinforcing materials were used: bamboo strips, aluminum strips, and nylon. Points were identified in each strip where the maximum tension was measured, and lines connecting these points were assumed to be the potential failure plane. No justification other than intuition is given for this assumption, and the tests were not carried to failure. The authors also conclude that the Rankine theory is adequate to describe the observed distribution of horizontal pressure.

Roth et al. (paper 5.33) describe studies motivated by the observed behavior of the excavations for the Los Angeles Metro in California. Very large loads were observed in the struts while tie-back anchors were not affected. Some gusset plates connecting the struts with the walers crimped, but wall deflections were not affected by the strut problem, and there were no measurable ground surface settlements, sidewalk cracks, or other signs of structural distress adjacent to the excavation. Several geotechnical experts were unable reach consensus on the cause of the problem.

The authors conducted a comprehensive analytical study of the behavior of strutted and anchored walls in stiff soils and rocks like those in Los Angeles. They used a finite difference computer program FLAC, which allowed realistic modeling of the construction sequence and of the non-linear soil properties. They considered various alternatives of struts, tiebacks, preloading, in situ stresses, and so on. They also investigated the possible effects of a "structural fuse," which limits the axial force that can be transmitted to a strut to prevent it from overloading.

The conclusions of the study were that, for competent soils like those at the site and in the Los Angeles basin in general:

- "The inherent stiffness of the struts attracts large support forces. This situation is further aggravated by the practice of strut preloading, and by horizontal tectonic compression of the region."

- "In contrast to struts, tie-back anchors are inherently flexible and, therefore, not susceptible to overloading. They allow excavation walls to deflect, regardless of the anchors' structural stiffness and the amount of preloading applied."
- "The amount of anchor preloading in a mixed Strut-Anchor support system has little, if any, effect on the support forces induced in the adjacent strut support levels."
- "Strut preloading, on the other hand, significantly increases the final strut loads, but only negligibly reduces excavation wall deflections."
- "Strut loads for excavations in competent bedrock without adverse bedding conditions can be significantly reduced by installing a 'structural fuse.' Induced support forces can be relieved at the expense of minor increases in wall deflections."

FLOW THROUGH POROUS MEDIA

Three papers presented to this session deal with flow into excavations, dewatering of excavations, or observations of quantities of flow. The field conditions are quite different in each of the cases reported.

Ergun and Nalçakan (paper 5.12) report on the design of a dewatering system for the excavation for a pumping station in Turkey. In order to design a well point system as accurately as possible, the engineers used the results of a pumping test to evaluate the in-place, large-scale permeability (hydraulic conductivity) that would affect the pumping operation. The authors point out that laboratory tests at small scale would not be appropriate because they tend to under-predict permeability for performance at field scale. This phenomenon is largely caused by the dominant effect of regions of high permeability that are likely not to be modeled in the laboratory sample. In fact, this has been illustrated by a number of recent investigations reported in the literature, including laboratory and field tests performed on the same soils. These studies show that fluid flow behavior is elated to the volume of the soil involved and hat values of hydraulic conductivity reach symptotic values at some finite but large haracteristic volume of soil.

The present project involved an excavation greater than 18 m (59 ft) deep with a cross section of 55 by 95 m (182 by 314 ft) at the bottom and 120 by 160 m (400 by 530 ft) at the top. The water table was about 1 m (3 ft) below the surface. The soils consisted of layers of clay, sand, and sandy clay. The predictions, using parameters derived from the field pumping test and the formulas for the case of a fully penetrating well, greatly over estimated the rate of discharge. The revised estimate using a partially penetrating well significantly improved the agreement between calculation and observation.

Castellanos and Sedano (paper 5.46) describe a case in which the observed flow into the excavation for a power plant in Mexico was significantly less than the predicted amount. A sump pump was used to control water flow into the excavation. The observed flow was 35 l/s (1.24 cfs), compared to a predicted flow of 470 l/s (16.6 cfs). This is a over prediction of better than one order of magnitude. The authors sate that the results of what are presumably laboratory permeability tests and field pumping tests showed about the same values of hydraulic conductivity, so they attribute the discrepancy between predicted and observed flows not to inaccuracies in predicting the values of permeability but to inaccuracies in the description of the stratigraphy around the site. This suggests that a more continuous evaluation of subsurface conditions, such as might be provided by a cone penetrometer, would have been useful in this case.

Shah et al. (paper 7.38) describe a technique for lining irrigation canals in India. The system consists of two outside shells of synthetic fiber fabric connected by nylon spacer threads. This is placed under water and the space between the fabrics is filled with a sand cement grout. The water-cement ratio is between 0.7 and 0.78, and the sand-cement ratio is between 1.5 and 2. The system is reported to work well, although the paper does not present data on the hydraulic conductivity of the liner or any other performance data.

GENERAL OBSERVATIONS

The papers presented in this session cover a variety of topics of importance in the design and construction of excavations. Most of them deal in one way or another with the behavior of braced or tied back excavations and concentrate on predictions, calculations, observations, and explanations of movements and earth pressures. The profession's understanding of these phenomena has obviously been significantly enhanced by the increasing use of field instrumentation, both to guide the process of construction and to improve the interpretation of predicted behavior. It is encouraging to note so many reported instances of combined instrumentation and analysis.

The General Reporter does have the impression that the details of the analytical models, especially the finite element analyses, may not be as well understood as the field instrumentation and the laboratory tests. In particular, many papers in this session report on comparisons between observed movements and calculations made with the program SOILSTRUCT or some of its descendants. The basic papers describing the use of this program, for example Mana and Clough (1981), make it clear that the stiffness of the soil is a critical parameter and that selecting its proper value is neither easy nor intuitively obvious. A more important limitation of this technology is that the numerical procedure used to simulate excavation has been demonstrated since 1970 to yield incorrect results for multiple steps of excavation, even for linearly elastic, isotropic, and homogeneous materials. Therefore, any agreement between calculation and observation for non-linear, inelastic, and inhomogeneous materials must be regarded as fortuitous at best. For a recent example in which the engineers have taken proper care for the details of the analysis as well as the numerous uncertainties in the soil behavior, the reader is referred to the recent paper by Whittle et al. (1993).

ADDITIONAL REFERENCES

Mana, A. I., & Clough, G. W. (1981) "Prediction of Movements for Braced Cuts in Clay," *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 107, No. GT6, pp 759-777.

U. S. Department of Labor (1989), Occupational Safety and Health Administration, *29 CFR Part 1926: Standards - Excavation*, Federal Register, Vol. 54, No. 209, 31 October.

Whittle, A. J.; Hashash, Y. M. A.; & Whitman, R. J. (1993) "Analysis of Deep Excavation in Boston," *Journal of Geotechnical Engineering, ASCE*, Vol. 119, No. 1, pp 69-90.

Wong, K. S., & Broms, B. B. (1989) "Lateral Wall Deflections of Braced Excavations in Clay," *Journal of Geotechnical Engineering, ASCE*, Vol. 115, No. 6, pp 853-870.

PAPERS PRESENTED AT THIS SESSION

5.03 Saran, Swami, & Khan, I. N. "Studies on a 4.0 m High Reinforced Earth Wall."

5.06 Tamaro, Mark; Lopez, Pablo; & Pamucku, Sibel "Prediction of Structural Slurry Wall Behavior."

5.08 Hohmeyer, D. W. "Augercast Pile Retaining Walls."

5.12 Ergun, M. U., & Nalçakan, M. S. "Dewatering of a Large Excavation Pit by Wellpoints."

5.13 Schick, T. "Reinforced Earth® Wall Supported by an Unstable Foundation."

5.14 Edstam, T., & Jendeby, L. "Behavior of a Braced Sheet Pile Wall in Soft Clay."

5.15 Rodgers, Richard, & Majchrzak, Michael "An Unsuccessful Urban Deep Excavation in Soft Soils."

5.16 Lim, Robin M., & Majchrzak, Michael "'Unconventional' Drilled Pier Underpinning."

5.18 Woo, Edwin P.; Soydemir, Çetin; & Liu, Thomas K. "Performance of a Pier Consisting of Three Sections."

5.21 Teparaska, Wanchai "Behavior of Deep Excavation Using Sheet Pile Bracing System in Soft Bangkok Clay."

5.22 Toyosawa, Yasuo; Horii, Noriyuki; & Tamate, Satoshi "Analysis of Fatal Accidents Caused by Trench Failure."

5.23 Rahimi, M. M.; Karwaj, C.; & Deb, P. K. "Failure of Sewerage Mains Constructed in Soft Estuarine Deposit."

5.25 Bray, J. D.; Deschamps, R. J.; Parkison, R. S.; & Augello, A. J. "Braced Excavation at the NIPSCO Bailly Station Power Plant."

5.26 Graham, J. R.; Humphries, R. W.; Fuller, J. M.; & Elliott, G. M. "Pressure Relief Tunnel System at US22/SR7 Interchange, OH."

5.30 Jamnongpipatkul, Pichit; Taesiri, Yongyuth; & Charumas, Voranit "Reinforced Soil Structure Test Sections in Mountainous Terrain."

5.31 Olson, R. E. "Failure of a Twenty-Foot High Retaining Wall."

5.32 Olson, R. E., & Heuer, R. E. "Failure of a Large Circular Excavation."

5.33 Roth, Wolfgang; Stirbys, Anthony; de Rubertis, Corbin; & Ellis, Richard "Performance of a Braced Excavation in Siltstone."

5.35 Abedi, H.; Porter, T. G; Lien, B. H.; & Ramos, J. A. "Behavior of Braced Sheetpile Excavation in Detroit Clay."

5.36 Marangos, Ch. "Underpinning of a Tilted Building, a Case History."

5.39 Horii, Noriyuki; Hanayasu, Shigeo; Toyosawa, Yasuo; Tamate, Satoshi; and Maruyasu, Takakazu "A Case History of the Collapse Accident of a Temporary Earth Support Structure."

5.40 Lin, Jeen-Shang, & Deng, J. G. "Predictions of the Behavior of a Deep Excavation."

5.44 Moh, Z. C., & Hwang, R. N. "Earth Pressures on Walls of a Deep Excavation."

5.45 Byrne, P. M.; Srithar, T.; & Kern, C. B. "Measurements and Predictions on the Elkhart Creek Culvert."

5.46 Castellanos, G., & Sedano, S. "Large Excavation Behavior at Petacalco, Mexico."

5.48 Lien, B. H.; Abedi, H.; Ramos, J. A.; & Porter, T. G. "Performance of a Semi-Rigid Braced Excavation in Soft Clay."

5.53 Pöttler, R. "Cut and Cover at Landrückentunnel North."

5.55 Parreira, A. B., & Azevedo, R. F. "Geotechnical Performance of a Tunnel in Soft Ground."

7.28 Matsui, T.; Nakajima, H.; Nagano, T.; Hosoi, T.; Fukuda, Y.; & Hayashi, K. "Filed Measurements of a Diaphragm Wall Foundation."

7.38 Shah, D. L.; Shrott, A. V.; & Parikh, Piyush V. "Lining of Perennial Canals under Flowing Conditions by Ulomat Grouted Mattress Technique."